# MURPHY CREEK RESERVOIR SHORELINE ASSESSMENT

A report to

B.C. HYDRO & POWER AUTHORITY

15-2-205

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This reservoir shoreline assessment report was prepared to present the results of a preliminary shoreline assessment and development of a preliminary safeline based on available information to June 1982.

Subsequent studies by B.C. Hydro to date, have included an update on the material included in this report based on available information to December 1982. The B.C. Hydro studies contained in the report entitled "Murphy Creek Project - Reservoir Shoreline and Proposed Protective Works" dated May 1983 should be read in conjuction with this report

May 1983

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# INTRODUCTION

## 1.1 PURPOSE

This report presents the results of an investigation and assessment of the shoreline for the proposed Murphy Creek Dam Project reservoir . The purpose of this work, designated as Phase I of a two phase project in Thurber Consultants Ltd. letter of February 10, 1981 to Mr. H. Taylor, was to:

i) confirm and expand on preliminary soil information provided by Mr. D. Cattanach of B.C. Hydro.

ii) confirm or more accurately define the recommended safeline distances provided in Thurber Consultants Ltd. letter of December 4, 1980 to Mr. H. Taylor.

iii) define specific areas which may require more intensive geotechnical consideration.

The scope and specific requirements of the work were discussed with B.C. Hydro and are summarized subsequently. Authorization to proceed with the work was received with B.C. Hydro P.O. 144-060.

Thurber Consultants Ltd. have been previously involved with the project by way of a terrain analysis for the Murphy Creek Reservoir area carried out during mid 1980. Also as noted in item (ii) above, a preliminary safeline study for selected areas of the reservoir was carried out during late 1980.

## 1.2 PROJECT DESCRIPTION

The following description covers only those items which are pertinent to this study.

The location of the proposed Murphy Creek Project is shown on Fig. 1-1. The damsite is located on the Columbia River, approximately 23 km downstream of the confluence of the Columbia and Kootenay Rivers, immediately upstream of Billy Creek, and opposite Oasis, B.C. The proposed project would create a 34 km long reservoir extending to the existing Hugh Keenleyside Dam on the Columbia River. An arm of the reservoir would extend 3 km up the Kootenay River to the Brilliant Powerplant.

Currently, a reservoir full supply level of 424 m is being considered. Relative to water levels surveyed during July 1981, this full supply level would raise the water levels during normal flow at the Hugh Keenleyside and proposed Murphy Creek dam sites by approximately 2 m and 12 m respectively.

To meet the Columbia River Treaty requirements, the Murphy Creek Reservoir would be drawn down each year between mid-January and early May. The amount by which the reservoir would be drawn down would depend on energy and downstream discharge requirements. The drawdown limit of the reservoir at the damsite would be 413.7 m.

The shoreline of the Columbia River has been inhabited for a long period prior to regulation. The population is concentrated in several communities which for the most part are located on the west side of the proposed reservoir.

Bridge crossings of the Columbia River by highways and CP Rail as well as several utility crossings are located within the reservoir area. Also, CP Rail trackage, highways, and various utilities run in close proximity to the proposed reservoir shoreline.

## 1.3 SCOPE OF STUDY

Initially the study was to include a shoreline assessment in built up areas around the proposed reservoir, specifically from km 2.5* to 16.5 (Robson, Pass Creek, Brilliant, Ootischenia) on the left bank, and km 7.0 to 19.8 (North and South Castlegar, Blueberry), km 22.9 to 25.5 (China Creek) and km 29.8 to 31.6 (Birchbank area) along the right bank. The study was also to include:

i) a definition of problems related to the CPR trackage and bridge crossing within the proposed reservoir area.

ii) investigation of any necessary protection requirements for the existing highway bridge crossing of the Columbia River.

iii) investigation of the groundwater regime in low lying areas of North Castlegar.

During the field investigation it became apparent that a significant amount of subsurface information could be collected for the entire reservoir area by surface mapping methods. Therefore, the scope of the study was changed by B.C. Hydro to include a shoreline assessment and a preliminary safeline location around the entire reservoir with the exception of km 0.0 to 4.5 on the right bank (B.C. Timber Pulpmill and Sawmill) which was to be studied separately by others.

Further into the study, we were informed by B.C. Hydro that an investigation of the groundwater regime in the Castlegar area was to be carried out by another consultant. At that time, 12 standpipe piezometers had already been installed to facilitate this investigation and Thurber Consultants continued to monitor the water levels in these piezometers.

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* Refers to distance measured downstream along the centreline of the Columbia River from the Hugh Keenleyside Dam.

The study was carried out assuming a reservoir full supply level of 424 m, with general comments concerning implications of 421 m and 427 m full supply levels.

The accuracy of the results of the study are commensurate with the mapping provided by B.C. Hydro.

## 1.4 INVESTIGATION PROCEDURE

The investigation procedure was carried out in several stages both in the field and in the office/laboratory.

The field portion of the investigation was carried out intermittently during the period from April 29 to November 7, 1981.* It consisted of sampling and installation of piezometers in 39 test holes, detailed mapping of 216 slope exposures, and sampling in 6 test pits.**

During the initial stages of the field work, the methodology of the study was altered somewhat because access for backhoe test pitting was not possible due to seasonally high river levels. The majority of the backhoe test pits initially planned were substituted by mapping and sampling of existing slope exposures.

The office/laboratory portion of the investigation was carried out as required during the field work. It consisted generally of a collection and review of available subsurface soil information in the reservoir area, in addition to laboratory testing of samples collected during the field program.

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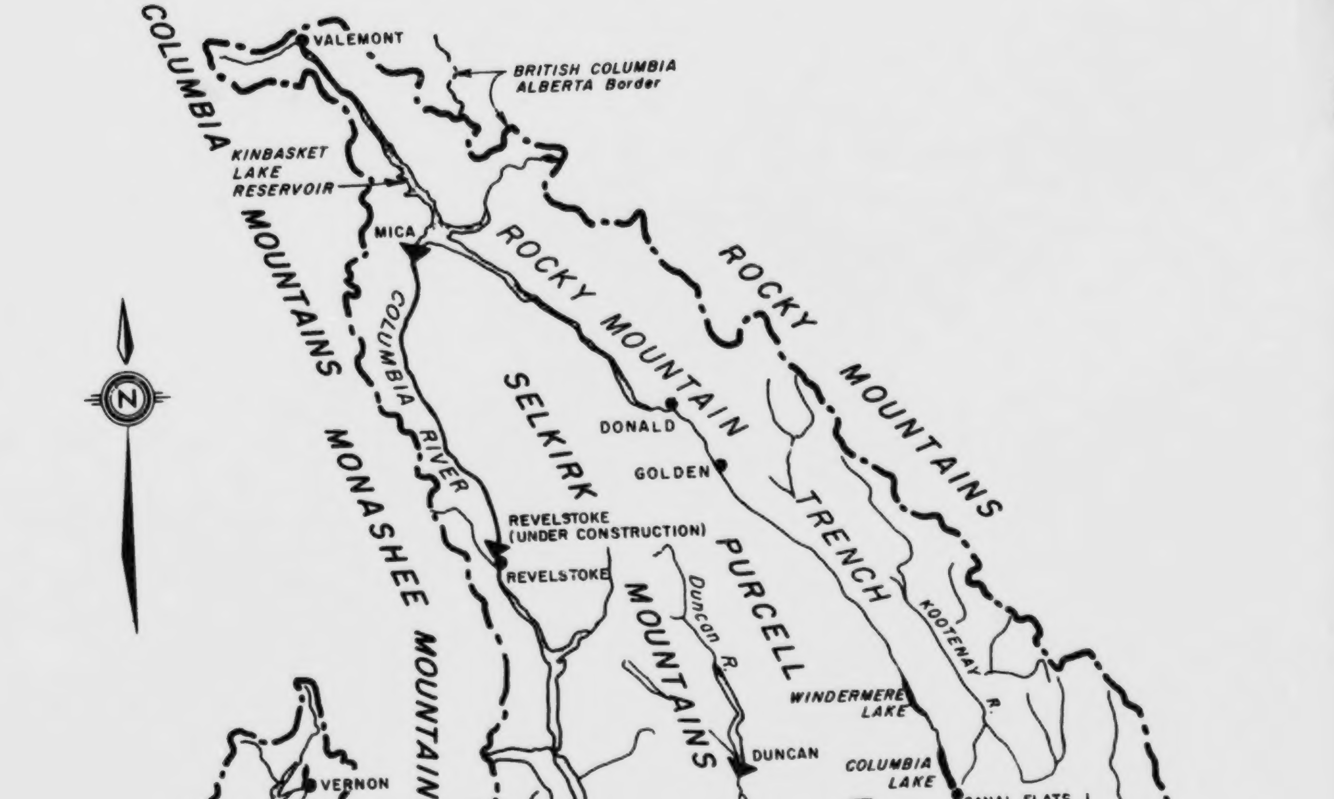
* Permission to enter private land was acquired by Mr. A. Lutz of B.C. Hydro's office in Castlegar.

** Mr. A. Chance of B.C. Hydro (summer student) assisted with slope mapping program.

Details of the investigation procedure are summarized in Appendix A.

The hydraulic studies were carried out under the direction of Dr. R. Kellerhals of Kellerhals Engineering Services Ltd.



NOTE: Base taken from British Columbia Hydro and Power Authority drawing no. 218-C14-B120 dated March 1981.

LOCATION PLAN

FIGURE 1-1

## Section 2

## SITE DESCRIPTION

### 2.1 GEOLOGY

The geology of the Columbia River Valley in the region of the proposed Murphy Creek Project is typical of the glaciated valleys of southwest British Columbia. Bedrock, glacial drift and recent alluvial deposits dominate the valley.

### Bedrock

The bedrock in the study area consists of high grade metamorphic and plutonic rocks including much granitoid gneiss (Little, 1962). The bedrock is often strongly foliated and jointed and structurally controlled bedrock terrain is evident in the uplands above the drift filled main valleys. Bedrock is intermittently exposed along the banks of the Columbia and Kootenay Rivers and has in many areas controlled the deposition of glacial outwash and recent alluvial deposits.

### Glacial History

The geological events prior to the last advance of the ice are not significant to the study since material deposited at that time has been either reworked or has been deeply buried beneath more recent glacial and alluvial deposits.

As a result of the most recent glaciation (Fraser) an irregular mantle of till was deposited upon the pre-existing rugged bedrock terrain and preglacial sediments. The ice then began to waste and degenerate into tongues in the Columbia and Kootenay River valley bottoms. Regionally and locally derived meltwater carried and deposited increasing amounts of sediment along the wasting margins of valley ice. Much of this outwash was sand with lesser amounts of gravel. These sediments, along with some glacial lake deposits, are preserved in bedrock defended locations high along the valley sides.

Outwash deposition under torrent conditions is suggested by the extreme size range of variably sorted sediments. Large boulders derived from glacial ice or nearby rock faces have been observed within deposits of finer grained stratified drift.

In the lower Pass Creek area extensive glacial lake silts were deposited in a lake dammed by ice in the Columbia Valley and fed by meltwater from above.

After the glacial ice had retreated from the valley bottoms, outwash deposition continued from upstream locations and tended to fill the valley bottoms. Deposition in the Columbia Valley was supplemented by sands and gravels deposited at the mouths of tributaries such as China Creek which may have carried sustained flows of meltwater.

In post-glacial time the Columbia and Kootenay Rivers eroded and reworked the mass of stratified drift to form a series of river terraces. Recent river erosion has exposed bedrock on the right bank in the area of the proposed Murphy Creek Dam.

## Recent Changes in Valley Morphology

The valley morphology has recently been affected by man's activities; the most notable of these are the construction of the Hugh Keenleyside Dam at the upper limit of the area and several highways in the Columbia Valley. Some stretches of the river banks, particularly upstream of Castlegar, have been riprapped. At a few locations in Castlegar and the small communities to the south, fill has been dumped down steep river terrace slopes resulting in shallow sloughing of surface materials on otherwise vegetated slopes formed of natural granular deposits.

The major change, however, is the effect of upstream regulation creating useable new valley bottom land and reducing river bank erosion.

## 2.2 SURFICIAL AND SUBSURFACE SOIL CONDITIONS

The following brief description of the soil conditions is based upon the investigations carried out to date (refer Appendix A). A terrain map showing the surficial geology of the area has been included in Fig. 2.1.

The surface and subsurface soil conditions vary depending on their relationship to the present river and its tributaries. In general the subsurface materials upstream of Castlegar are finer grained than the materials downstream.

In the low lying flat areas approximately 5 to 10 m above the present river level the materials are generally fine silty sands, probably overbank deposits (deposits laid down during flood conditions). The deposits above el 450 - 470 m are kame terraces consisting of coarse sandy gravelly materials. In areas below el 450 - 470 m, and underlying the overbank deposits, outwash deposits are present consisting mainly of fine gravelly sand. Within these sediments, interlayered sands and gravels and possible occasional boulder pavements, were encountered.

Exceptions to the preceding generalizations occur in areas of existing creeks where fan deposits consist of coarse sandy bouldery gravel. In the Pass Creek area however, the fan deposits are primarily fine sands and silts.

## 2.3 EXISTING RIVER ARMOUR

Recent river processes, particularly those prior to upstream regulation have produced a natural armour of gravel and cobbles lining the river banks. This armour increases in size and thickness where exposure to erosive river forces increases, i.e. outside downstream and inside upstream segments of bends in the river, and where the velocity of the river increases due to an increased gradient and/or decreased cross sectional area.

The Columbia River upstream of the ferry crossing or the rock bluffs locally known as "The Gates" is a transitional feature between the lower Arrow Lakes proper and the Columbia River, and as such has certain features characteristic of both lake and river topography. In this area the river gradient is relatively flat, with well developed beach slopes (of relatively fine materials) along the shoreline and contrasts with the river eroded slopes (naturally armoured) further downstream.

In the upper 20 to 25 km of the reservoir area, the proposed full supply level of 424 m and projected spillway design flood level are below that which the river banks have experienced during unregulated floods in the past. Thus the existing river armour is particularly significant in this area. A summary of the river armour details observed at various locations around the reservoir is given in Appendix A (Tables A-1 and A-2).

## 2.4 SLOPE STABILITY

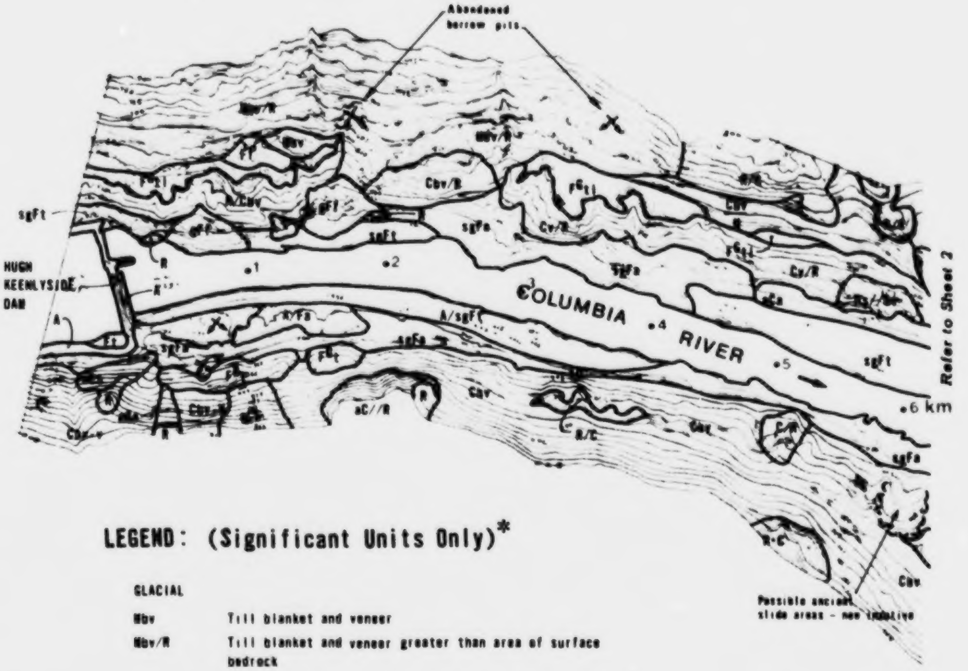
Shallow sloughing and ravelling of granular materials on steep river slopes is common in the river valley particularly in that portion south of the Columbia - Kootenay River confluence. Many of these slopes are oversteep, (defined as steeper than 1.5H:1V for the purpose of this study) and are subject to surface sloughing. It was observed that where surface sloughing occurred, the slopes were generally 1.5H:1V or steeper. As such they presently pose a safety hazard to some facilities near their crest.

No deep seated slides were encountered by the field investigations.

## 2.5 GROUNDWATER

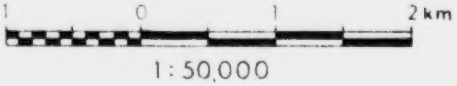
The piezometers, installed in each of the test holes during the field program, have not been in place for a sufficient length of time to monitor a complete yearly groundwater regime cycle. A record of the water level measurements to date taken in each of the piezometers is given in Table A-3 in Appendix A.

Based on the piezometer measurements obtained to date and general river level information provided by the operators at the Hugh Keenleyside Dam, it is apparent that the gradient of the groundwater surface is fairly flat - possibly 2 to 3% in the Castlegar area. This observation is based on piezometer readings taken during the drier portions of the year. The piezometer gradient may be steeper during spring thaw when it is expected that groundwater recharge due to surface water infiltration would have a significant contribution to the groundwater regime.



## LEGEND: (Significant Units Only)*

| | |
|---|---|
| **GLACIAL** | |
| Mbv | Till blanket and veneer |
| Mbv/R | Till blanket and veneer greater than area of surface bedrock |
| **FLUVIOGLACIAL** | |
| $sgF^{G}_{t}$ | Sandy-gravelly fluvioglacial terrace |
| $F^{G}_{tl}$ | Level surface of fluvioglacial terrace |
| $F^{G}_{s}$ | Steep erosional slope on fluvioglacial terrace |
| $F^{G}_{ts}$-V | Steep fluvioglacial terrace slope - gullied |
| **GLACIOLACUSTRINE** | |
| zLt-V | Silty lacustrine terrace-- gullied (Pass Creek area) |
| **FLUVIAL** | |
| sgFt | Sandy-gravelly and coarser fluvial terrace |
| Fts | Steep erosional slope on river terrace |
| gFf | Gravelly and coarser fluvial fan |
| gsFt | Gravelly sandy fluvial terrace |
| $gF^{A}_{l}$ | Gravelly and coarser active floodplain deposits |
| Fbv R | Fluvial blanket and veneer equals area of exposed bedrock |
| **BEDROCK AND COLLUVIAL** | |
| R | Bedrock |
| Rs | Steep bedrock |
| R//C | Areas of exposed bedrock much greater than areas of colluvium |
| $R/\frac{C}{M}$ | Areas of bedrock greater than areas of colluvium overlying till |
| aCa | Colluvial apron formed of angular boulders |
| **ORGANICS** | |
| Obv | Organic blanket and veneer |
| **ANTHROPOGENIC** | |
| A | Man emplaced or man-modified surface materials. Typically large highway fills |

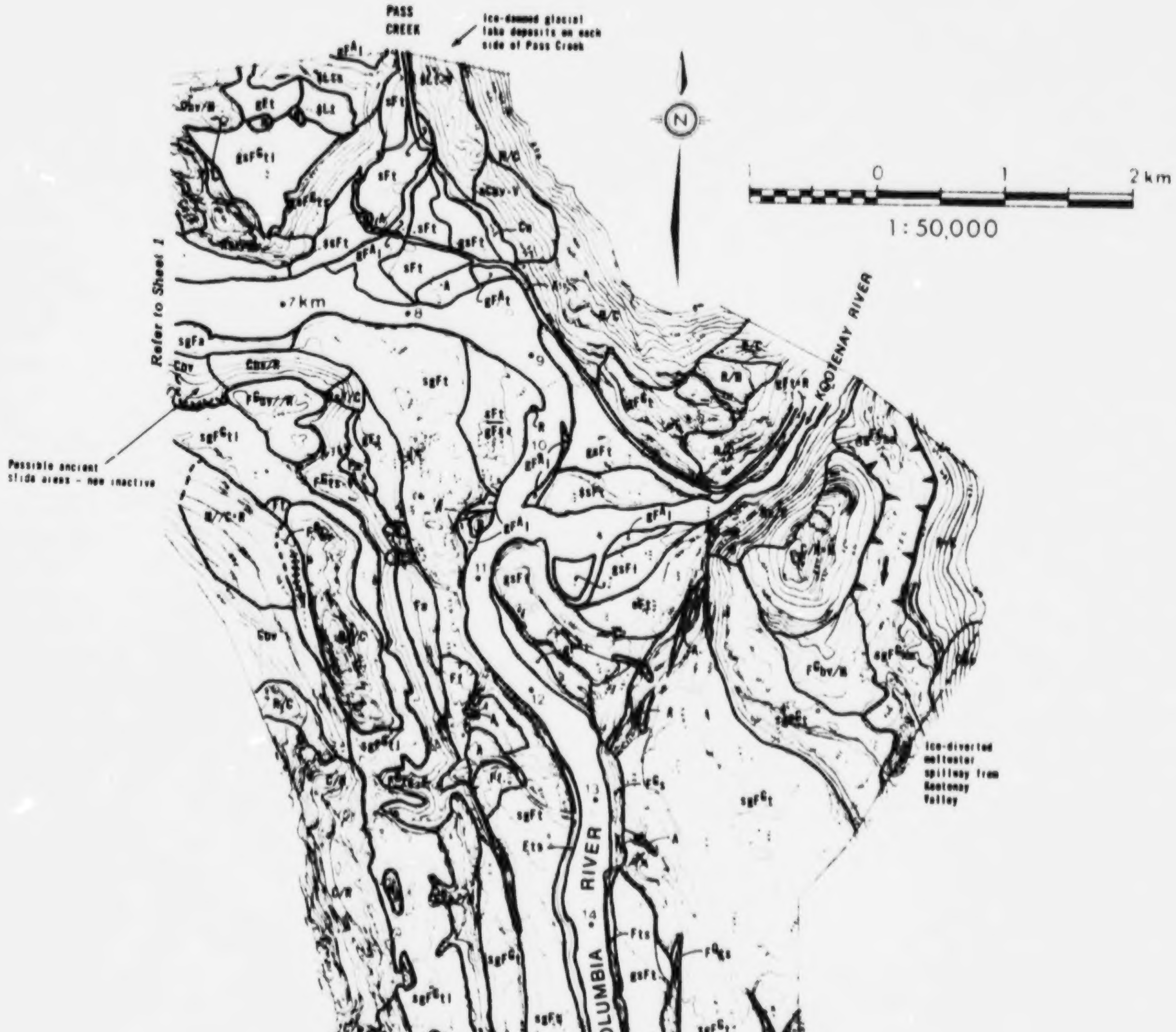


*NOTE: Figure taken from Thurber Consultants Ltd. Drawing No. 15-2-171-1 dated July 1980

## TERRAIN MAPPING
Sheet 1

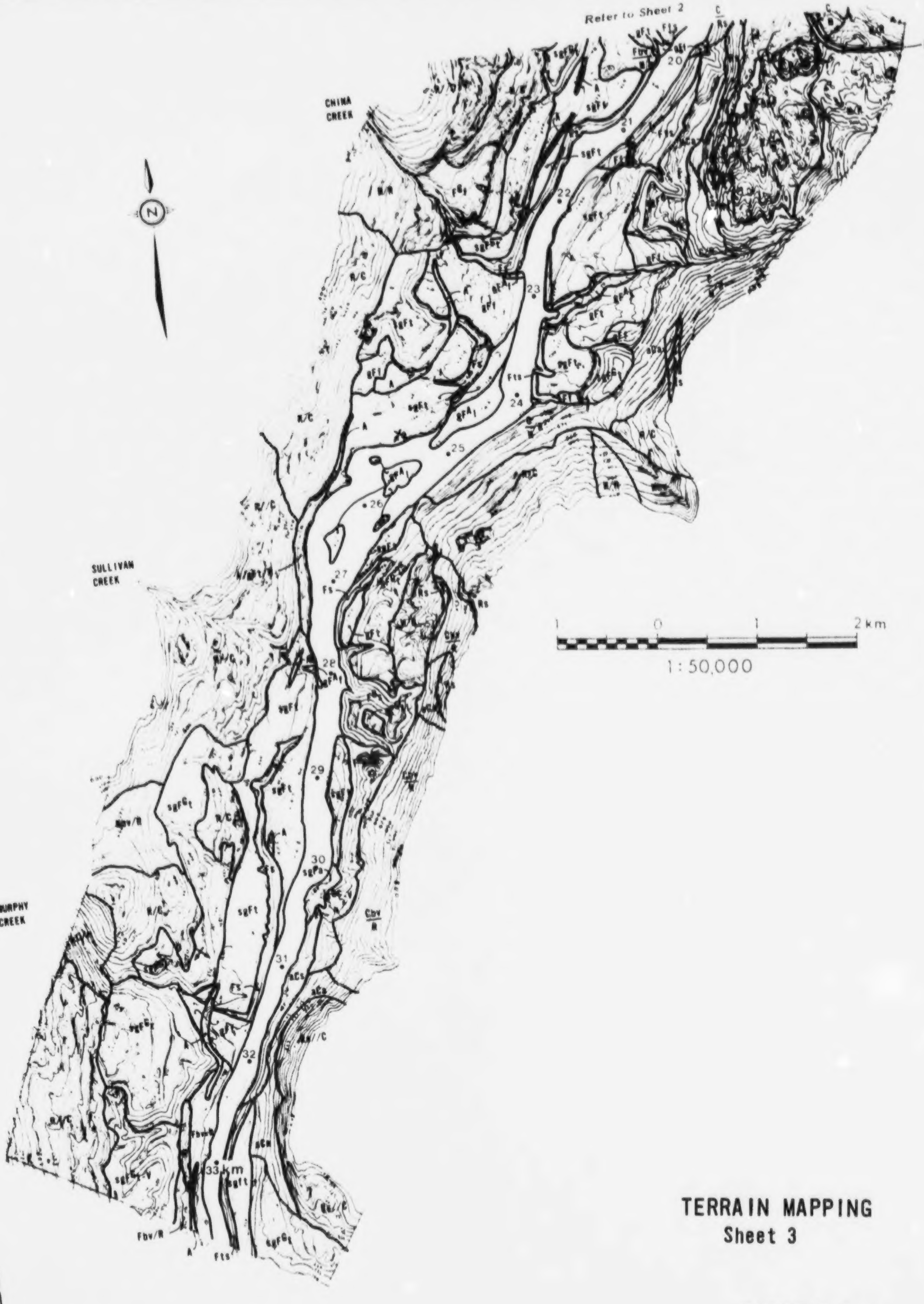
THURBER

**FIGURE 2-1**


PASS CREEK
Ice-dammed glacial lake deposits on each side of Pass Creek
N
0
1
2 km
1:50,000
Refer to Sheet 1
•7km
•8
9
10
11
12
13
14
15
16
17
18
19
KOOTENAY RIVER
COLUMBIA RIVER
Possible ancient slide areas - now inactive
Ice-diverted meltwater spillway from Kootenay Valley
BLUEBERRY CREEK
Refer to Sheet 3
sFt
sgFt
gFA
Cbv
Fts
R/C
A

# TERRAIN MAPPING
Sheet 2

FIGURE 2-1



## TERRAIN MAPPING
Sheet 3

FIGURE 2-1

## Section 3

## PROPOSED RESERVOIR OPERATION AND ITS EFFECT

### 3.1 RESERVOIR OPERATION

B.C. Hydro have selected a full supply level for the proposed reservoir of 424 m. Consequently with normal flows the water level would be raised by approximately 12 m at the proposed dam and 2 m at the Hugh Keenleyside Dam (cf. profiles 1 and 2 on Fig. 3-1).

The FSL refers to a level during normal operating conditions measured at a point near the Robson-Castlegar ferry crossing. The reservoir would be narrow with a small storage capacity, thus the flow through the reservoir would result in a slight gradient of the water surface. Profile 2 on Fig. 3-1 illustrates this gradient during normal operating conditions.

The flood discharge through the project area is reduced by existing upstream regulation. Profile 3 on Fig. 3-1 depicts the projected water surface for the 200 year spillway design flow. To permit comparison, two additional profiles have been added to Fig. 3-1; profiles 4 and 5 depicting river levels for unregulated (pre 1963) 50 and 200 year floods respectively.

Predicted channel velocities resulting from a backwater study carried out by B.C. Hydro are shown in Table 3-1. It is evident that the velocities through the reservoir will still be appreciable, although much reduced from those presently and previously experienced.

In late winter and early spring, the Arrow Lakes reservoir is drawn down to accommodate late spring run-off. In order to draw the Arrow Lakes reservoir down sufficiently, the Murphy Creek reservoir would also be drawn down. The simulated regulated level of the proposed reservoir (at the dam site), prepared by B.C. Hydro on the basis of 1940 to 1980 hydrological data, is summarized on

Fig. 3-2. The figure indicates a maximum mean seasonal drawdown of about 5 m. Daily fluctuation of the reservoir water level at the proposed Murphy Creek dam site would be limited to a maximum of 2 m during March and April of each year and 1.5 m during other months. Daily fluctuations upstream of the dam would be slightly less, depending on location and inflow. The maximum daily fluctuation would not occur on a regular basis. For the purposes of this report, a maximum daily fluctuation of 1 m is considered appropriate. The rate of daily drawdown would not exceed 1 m/h.

## 3.2 GENERAL EFFECT OF THE PROPOSED RESERVOIR ON THE SHORELINE

Much of the reservoir shoreline is steep and thus the reservoir would not, for the most part, inundated large areas. However, there are several small areas that would be affected which are developed and in some cases populated.

For a FSL of 424 m the total length of the shoreline would approximate 85 km. Inundated areas would include residential land in the lower areas of the communities of North and South Castlegar and China Creek; recreational land in the areas of Pass Creek and Birchbank; and farmland in the vicinity of Brilliant, southern Ootischenia and Champion Creek. Also, the bases for at least two overhead utility crossings of the Columbia River may be innundated in addition to 4.7 km of rail grade in the Birchbank area, portions of the ferry crossing aprons in north Castlegar and Raspberry, and poles for some portions of overhead utility lines running parallel to the shoreline near Genelle.

The present river channel is for the most part adequately armoured for present river conditions. However once the water level is raised, new areas would be exposed to wave action resulting in some beaching and regression of the shoreline. Also some bank erosion could still occur from currents which would exist (although reduced from present conditions) within the reservoir. The amount of shoreline regression would depend on many variables (discussed in Section 4 and Appendix B) and could affect existing and future land uses close to the shoreline. These include residential land in North and South Castlegar, Blueberry and China Creek; cemeteries in Brilliant, Ootischenia and the Champion Creek area; roadway grades in Robson; railway grades near Genelle and Blueberry; and the existing CPR and highway bridge abutments. Shoreline protection could be provided, where necessary to protect these facilities.

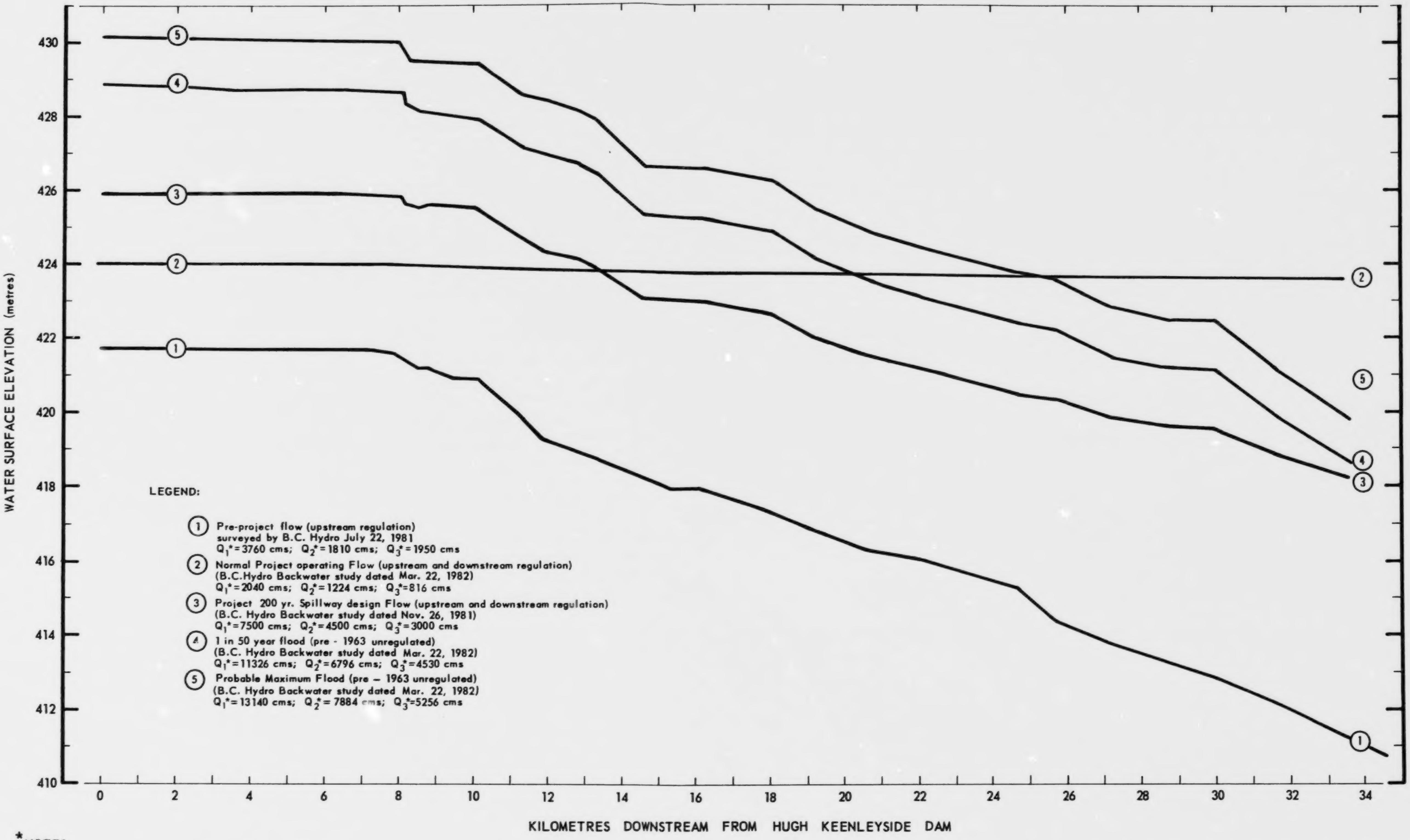
The Columbia River, over the upstream portion of the reservoir, has experienced water levels higher than the Normal Operating Levels planned for the Murphy Creek project. However, these levels have been the result of abnormal run-off and in some areas have not been of sufficient duration to allow the river regime to accommodate such conditions. The river regime would be modified by the Murphy Creek project because of sustained high water levels (although generally smaller flows than previously experienced at these levels). The natural armouring, for example, may not be adequate to withstand wave erosion at these higher levels. Further, raising the river level for a sustained period will affect the groundwater table. The effect of raising the groundwater table on basements and utilities is being studied separately by B.C. Hydro.

## TABLE 3-1

## RESULTS OF BACKWATER STUDY
## BY B.C. HYDRO
(Nov. 1981, March and April 1982)

| Flow Conditions (cms)* | Channel Velocity (m/s) | | |
|---|---|---|---|
| | Average | Maximum | Minimum |
| 2040<br>(mean annual flow; post Murphy) | 0.46 | 0.83<br>(km 11.8) | 0.19<br>(km 25.4) |
| 7500<br>(200 yr spillway design flow; post Murphy) | 1.87 | 3.14<br>(km 31.7) | 0.70<br>(km 2.1) |
| 2040<br>(mean annual flow; pre 1963) | 1.34 | 2.33<br>(km 10.1) | 0.27<br>(km 2.1) |
| 11326<br>(50 yr flood; pre 1963) | 2.40 | 4.41<br>(km 31.7) | 0.90<br>(km 3.6) |
| 13140<br>(Probable Maximum Flood; pre 1963) | 2.52 | 4.67<br>(km 31.7) | 0.91<br>(km 7.9) |

*Total flow dowstream of Columbia River & Kootenay River confluence.



*NOTES:

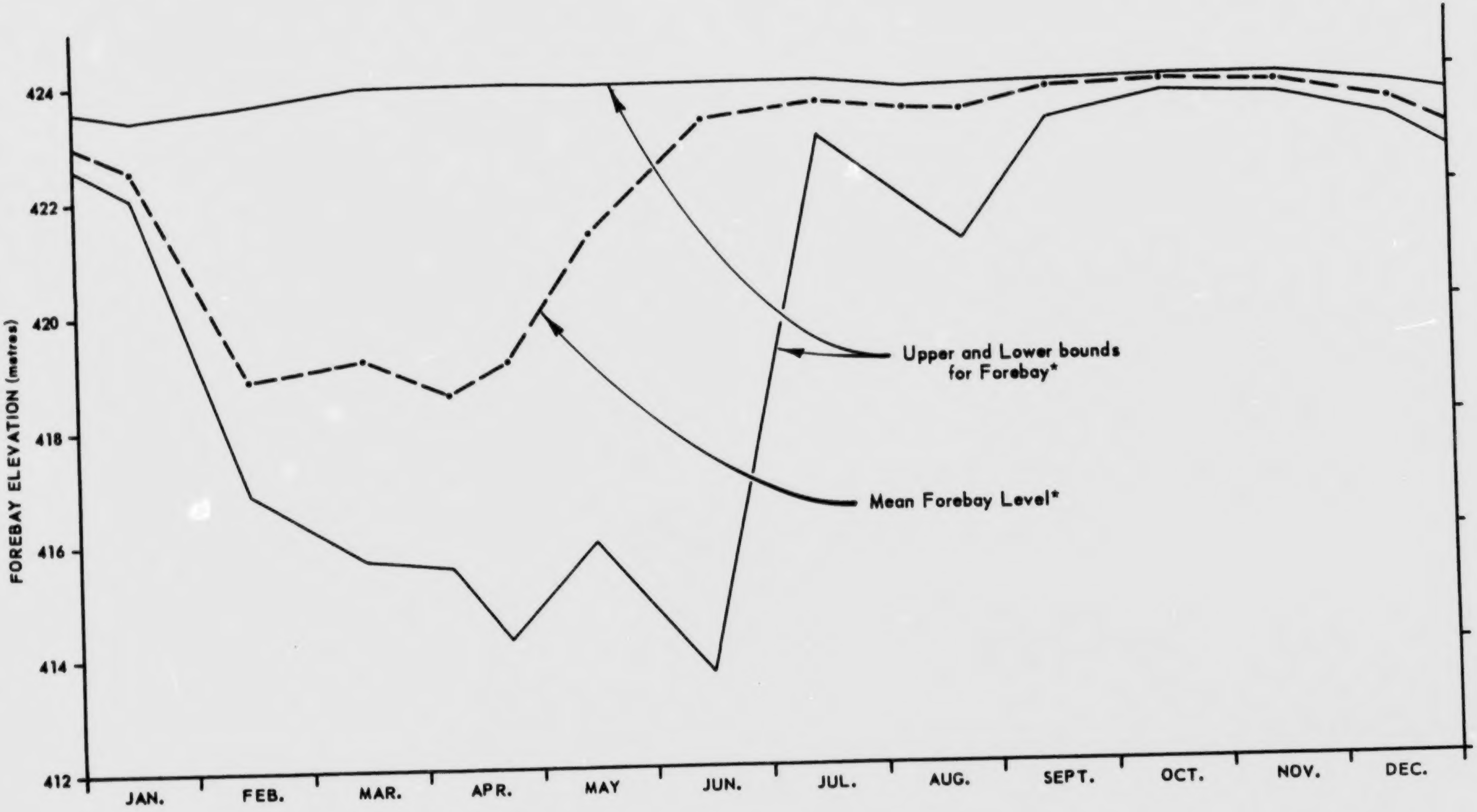
$Q_1$ refers to total flow downstream of Columbia River – Kootenay River confluence

$Q_2$ refers to flow in Columbia River upstream of confluence

$Q_3$ refers to flow in Kootenay River upstream of confluence

SUMMARY OF WATER LEVEL PROFILES WITHIN THE RESERVOIR AREA

FIGURE 3-1



* Information provided by B.C. Hydro Generation Planning Dept. computer printout dated Sept. 16, 1981 (projected on basis of 1940 to 1980 data)

SIMULATED REGULATED LEVEL
OF MURPHY CREEK RESERVOIR
AT PROPOSED DAM SITE

FIGURE 3-2

## Section 4

## ENGINEERING ANALYSIS AND ASSESSMENT

In the previous section of this report the operation of the proposed Murphy Creek reservoir has been discussed, and possible effects of the reservoir have been briefly identified. This section is concerned with geotechnical evaluation of the reservoir shoreline area and includes a detailed discussion of the effect the reservoir will have on existing slopes.

### 4.1 SLIDE POTENTIAL

The soils along the proposed reservoir shoreline are generally non-cohesive, granular silts, sands and gravels or mixtures thereof (refer Fig. 2-1 of Section 2).

The soils are of medium to high permeability (K-value greater than about 0.01 cm/sec) particularly in areas where water level fluctuations will be most pronounced. With soils of this permeability the likelihood of slumping due to normal drawdown of the reservoir, as described in Section 3.1, is considered to be very low. Shallow sloughing of the steeper banks could occur under emergency rapid drawdown conditions.

Flooding of the toe of steep slopes by reservoir impoundment tends to reduce their stability due to uplift. However, because of the granular nature of the soils along the reservoir shoreline, it is unlikely that any significant instability would be initiated.

Some ancient landslide areas are visible above the highway on the right bank immediately downstream of the pulp mill area (Fig. 2-1). These slides are no longer active and they do not appear to pose any hazards at present or with the proposed Murphy Creek project.

Some of the existing river banks presently show evidence of shallow sloughing and ravelling (Section 2.4).

## 4.2 SHORELINE REGRESSION

The major components contributing to shoreline regression are beaching and to a lesser extent reservoir current erosion. As noted in Section 4.1, the likelihood of large scale slumping or sliding due to filling and normal drawdown of the proposed reservoir is considered remote, and is therefore not included as contributor to shoreline regression.

### Beaching

Shore erosion due to wind generated waves (possibly contributed to by the passage of boats) is herein referred to as beaching.

The major factors that determine the amount of shoreline regression due to beaching of a given slope are wave energy (height, period, velocity, etc.), reservoir operation, and the gradation and grain size of the materials forming the reservoir banks.

Fig. 4-1 illustrates the ultimate effect of beaching. As a beach slope forms, the back slope is eroded by undermining and intermittent sloughing. Eventually an essentially stable beach slope develops and regression of the shoreline is virtually halted.

It should be noted that on Fig. 4-1, no allowance has been made for redeposition of material eroded during the process of beaching. Due to possible current erosion within the reservoir it is conservatively assumed that this material cannot be relied on to accumulate and form part of the beach slope. Therefore it has been neglected except for the assumption that it will heal beaching which occurs below the level noted for beach slope development.

Consideration of the duration of wave generating winds together with the proposed reservoir operation (refer Section B.1 of Appendix B) indicates that beaching will be most significant within a meter or so of FSL and that at the lower levels it can be ignored.

The elevation range over which the beach slope is assumed to develop is shown on Fig. 4-1 to be between 2 m below and 1 m above the FSL. This 3 m range recognizes that the majority of beaching will occur while the reservoir level is at the FSL or 1 m below, and includes an allowance for wave run up and an allowance for 'bottom-break'. Bottom-break covers the energy that is transmitted to the slope as a wave begins to feel the bottom, which occurs at a depth equal to about 1.5 times the significant wave height. Preliminary calculations indicate the significant wave height would be about 0.7 m (refer Section B.2 of Appendix B) thus resulting in an allowance of about 1 m for bottom-break. The allowance for run up varies with the slope angle and slope material. For computation of beaching effects and the location of the preliminary safeline, a run up allowance of 1 m was assumed throughout most of the reservoir (refer Fig. B-3, Appendix B). Some shoreline sections where the slopes are as flat as 10 on 1 or less, or would be sheltered or cut off from the main reservoir, would remain virtually intact and would not regress.* Of the total 85 km of reservoir shoreline, it is expected that 58 km would experience regression by beaching.

A relationship between grain size and beach slope is presented on Fig. 4-2. This relationship is based on extensive studies and documentation of beach slopes in lakes and other reservoirs. The curves designated 'limited' and 'moderate' exposures correspond to fetches on the Arrow Lakes in the order of 2 and 5 km respectively. In Section B.2 of Appendix B, the calculated average fetch for the proposed Murphy Creek Reservoir is 800 m. The beach slopes for various grain sizes assumed in the analysis are tabulated on Fig. 4-1 and are also depicted on Fig. 4-2.

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* The location of these special areas are detailed in Section 4.3, page 18.

The grain size of the material which would ultimately determine the beach slope was obtained from the data gathered during investigation and examination of slope exposures around the reservoir. Medium to fine sands dominate the soils around the reservoir. However the gravels presently interlayered with the sand in most areas would govern the beach slope. The estimated governing grain size at many of the locations examined is summarized in Tables A-1 and A-2 in Appendix A.

Shoreline regression due to beaching normally occurs slowly and could take many years to reach the predicted ultimate extent.

### Reservoir Current Erosion

Shore erosion due to the current within the reservoir is discussed in Section B.3 of Appendix B.

With the exception of a 0.8 km section along the right bank of the Columbia River between km 10.8 and 11.6 (approximately 2 km upstream of the highway bridge crossing of the Columbia), current erosion in the proposed reservoir will not be a major contributor to shore line regression.

Along the right bank upstream of the highway bridge, the slope which would form the bank of the reservoir is not sufficiently armoured to withstand flows during flood conditions. Existing minor sloughing and sliding within this section has likely been initiated by erosion during previous relatively high river flows.

In areas other than that noted above, from the Hugh Keenleyside Dam to km 23.1 and 22.8 on the left and right bank respectively, the bank material is of sufficient grain size to withstand reservoir current erosion. In some cases the material is present as an existing river armour from past flooding, and in other cases, the bank material that would make up the future beach slope would be of sufficient size to resist current erosion.

In areas downstream of km 23 to the proposed Murphy Creek dam site, there is a potential for erosion. This current erosion would not be significant, however, for when the reservoir levels are high the erosive capacity of the currrent is minimal. The erosive velocities occur during periods of drawdown due to flood conditions, specifically, during the spillway design flood, at which time the water level would be drawn down below the profile for a pre 1963 50-year flood (see Fig. 3-1).

## 4.3 APPLICATION OF THE SAFELINE CONCEPT

### The Concept

The "safeline" is a conservatively located line beyond which the security of residents and residential improvements can be reasonably assured. Where beaching is the prime consideration, the safeline is established at the predicted maximum extent of shoreline regression. For this study groundwater considerations such as basement flooding potential were not taken into account in locating the safeline. This terminology was established for the Arrow Lakes Development in cooperation with B.C. Hydro's Properties Divison.

The purpose of this study was to establish a preliminary safeline. The degree of accuracy in locating the safeline was dependent primarily on two factors.

i) the extent of investigation in particular areas (some areas were heavily vegetated and detailed surface inspection was not possible; other areas were drilled, or exposed slopes were mapped, and the subsurface soil conditions were well established.)

ii) the availability of large scale mapping (In areas where mapping was available at a scale of 1 inch = 100 ft, with contour intervals of 1.5 ft, the degree of accuracy was generally greater than that in areas where mapping was available only at a scale of 1:5000, with contour intervals of 5.0 m.)

### Basis for Locating the Preliminary Safeline

The safeline was located by applying the procedure shown on Fig. 4-1 to the banks around the reservoir. Where the existing slopes were flatter than the estimated beach slope, or where bedrock or suitable riprap was present (and slopes above were at a stable configuration), the safeline was established at the minimum freeboard elevation.

Except in cases noted subsequently the following minimum freeboard allowances have been utilized for this study (refer Section B.2, Appendix B):

- for roads, railways and other industrial facilities; 1.5 m above FSL or 1.0 m above spillway design flow level whichever is greatest (see Fig. 3-1).
- for residential areas; 0.5 m added to the above.

The minimum freeboard allowances were reduced by 0.5 m for the following two conditions:

i) where the existing slopes at FSL are very flat (flatter than 10 on 1); areas where these conditions exist include:

- left side of Pass Creek from km 0.75 to 0.50 (that portion of the Pass Creek delta south of highway).
- right side of Columbia River from km 8.75 to 10.20 (low area of North Castlegar).
- right side of Kootenay River from km 1.15 to 0.75 (low area in Brilliant).
- left side of Kootenay River from km 1.50 to 0.90.

ii) where low lying areas (at or below the FSL) would be sheltered or cut off from the main reservoir body; areas where these conditions exist include:

- left and right sides of Pass Creek upstream of km 0.75 (portion of Pass Creek delta upstream of highway).
- left side of Kootenay River opposite km 1.00 to 0.50 (narrow oxbow lake feature).
- inside of island on right side of Columbia River opposite km 10.45 to 10.70.
- left side of Columbia River opposite km 28.65 to 29.70.

The potential flooding of basements and utilities was not a consideration in locating the safeline in this study.

## 4.4 LOCATION OF PRELIMINARY SAFELINE

The preliminary safeline location for a FSL at 424 m and no shoreline protection is provided on Fig. C-2 (sheets 1 through 10) in Appendix C, and tabulated at 0.25 km intervals in Tables 4-1 and 4-2 for the left and right banks respectively.

On the right bank from km 10.8 to 11.6, where reservoir current erosion is of concern, no safeline has been provided since a setback to allow for current erosion can only be reliably determined with the aid of model studies. Other areas where no safeline has been provided are along the left bank from km 19.00 to 19.95 and from km 31.55 to 32.45, where subsurface conditions are uncertain.

Included in the right column of Tables 4-1 and 4-2 are remarks relating to shoreline structures and developments. For the most part, the remarks are self explanatory. Remarks pertaining to river bridge crossings and utilities are clarified in subsequent sections.

In calculating the safeline location without shoreline protection, it has been assumed that existing shoreline conditions do not change significantly, i.e. no road grade or rail grade reconstruction, and no fill construction or dyking in low areas.

## 4.5 SHORELINE PROTECTION

The areas to be considered for protection were selected to meet the following objectives:

i) to preserve major facilities such as bridge crossings, Castlegar sewage lagoon, etc.

ii) to reduce setback distances for the safeline which would otherwise impinge on nearby residential and other developments, structures, utilities, etc.

Tables 4-3 and 4-4 are arranged in a manner which, working from left to right, identifies the location of the protected section, summarizes estimates of volumes of material required to provide minimum protection, and provides brief comments pertaining to the protected section. Continuous sections have been divided further in order to isolate the purpose for protection (i.e. roadway, buildings, rail grade, etc.) and/or variation in estimates of volume of material per metre of length. The tables have been divided in this manner to facilitate further studies.

The sections of shoreline selected for protection total 22 km.

Protection requirements are discussed briefly in Sections B.2 and B.3 of Appendix B. A conceptual design for protective works is shown on Fig. 4-3. As well graded a material as is practical to obtain should be used to construct the protective works. Not only are these materials necessary to satisfy filter requirements, but they are inherently more stable under normal wave conditions. They also tend to "pave" under storm conditions. With "paving" the finer particles are removed leaving the larger particles and thus the $D_{50}$ size increases. Quantity estimates provided in Tables 4-3 and 4-4 are based on Fig. 4-3.

In preparing a conceptual design for the minimum protection requirements, the logistics of providing protection have not been considered. For example, an evaluation of whether the protected section should be wider to accommodate vehicular traffic, or whether protection should be continuous over a long area rather than having intermittent protected areas, etc. was not considered.

Shoreline regression due to beaching will occur slowly over a number of years. In some areas it would be advantageous to observe shoreline regression and beach slope development prior to placement of any protective material. Areas where this may be of particular benefit include sections of the highway grade along the left bank from km 4.10 to 7.20 (the existing fill, although not investigated in detail, may provide suitable protection for the road grade).

Sources of protection material have been noted in the fans of Blueberry, China and Murphy Creeks, however no testing was carried out during this study to determine the adequacy of such deposits with regard to gradation and volume requirements. Riprap could be made available from proposed rock excavations at the Hugh Keenleyside and Murphy Creek dam sites. The adequacy of these sources needs to be further studied.

It is important to note that protective measures of this type will not stabilize slopes that are presently oversteep and are likely to regress for reasons totally unrelated to the project Regression of an oversteep slope can only be prevented by reconstruction of the slope to a stable configuration by careful filling and flattening or by constructing retaining walls. Areas where the slopes are oversteep are identified on Fig. C-2.

## 4.6 EFFECT OF PROPOSED RESERVOIR ON CPR TRACKAGE

The CPR trackage as effected by the proposed reservoir is the subject of a separate study by Thurber Consultants. The problem is defined and briefly discussed below.

Of the 36 km of trackage within the reservoir area, 4.9 km of the grade would be threatened by regression of the reservoir shoreline, and an additional 4.7 km would be inundated. In addition to the rail grade, a 214 m long bridge structure across the Columbia River at km 8.1 may be affected and a 30 m long bridge across Murphy Creek (km 31.80) would be inundated. The locations of specific sections of the rail trackage that would be affected by the proposed reservoir are noted briefly in Tables 4-1 and 4-2.

Shoreline protection would eliminate the threat of erosion of the rail grade. However, along the right bank, from km 27.95 to 32.95, the rail grade would have to be raised and/or relocated above minimum freeboard elevation.

The existing CP Rail bridge crossing of the Columbia River may require additional abutment protection. Scour around the existing river piers has occurred in the past, and repaired on an "as required basis". Any continuing scour would be at a reduced rate due to generally decreased water velocities in the reservoir.

The existing Murphy Creek bridge structure, with a timber deck at about el 422 m, would need to be replaced to raise the trackage above minimum freeboard elevation. It is possible that the same structure, a steel girder bridge, could be relocated on new abutments. Alternatively the bridge structure could be replaced or a large diameter multiplate culvert structure could be constructed.

Grade relocation details (including volume and cost calculations), details concerning the Columbia River bridge crossing (structural integrity of the bridge, hydrological studies) and details concerning replacement of the Murphy Creek bridge (type of structure, hydrological studies) are being covered in the separate study.

## 4.7 EFFECT OF PROPOSED RESERVOIR ON EXISTING HIGHWAY BRIDGE CROSSING

Fig. 4-4 shows the details of the existing 365 m long highway bridge crossing of the Columbia River at km 13.2. Although no contact has been made with the designer of the structure, the bridge design drawings indicate a Murphy Creek Project with a FSL of 430 m was taken into account during the bridge design.

The present concrete apron protection of the abutments should be examined in detail for their adequacy to withstand exposure to the sustained high water levels of the proposed project. It may be necesary to place additional protection material to prevent undermining the concrete aprons and thus ensure continual stability of the bridge abutments.

## 4.8 EFFECT OF PROPOSED RESERVOIR ON EXISTING UTILITIES

The locations of the existing 13 utility and cable crossings of the Columbia and Kootenay Rivers within the proposed reservoir area (excluding bridge and ferry crossings) are provided in Table 4-5. Included in the table are elevations of the tower bases adjacent to the rivers (as provided by B.C. Hydro), and remarks concerning the location of the towers relative to the safeline (unprotected) for the proposed Murphy Creek Project.

In most cases, the effects of the proposed reservoir require no further clarification; the towers either would not be affected, or would fall below the safeline and require protection. Crossings which require further assessment include:

i) the left tower for the gas pipeline crossing at km 9.35 where slope regression is presently encroaching on the tower anchors.

ii) the right tower for the above gas pipeline crossing, and both the left and right towers for the telephone crossing at km 10.15 where the tower foundations would be partially or wholly submerged during flood flows.

In addition to the utility crossings, there are cases where utilities following the existing river banks would fall within the safeline of the proposed Murphy Project. These cases include:

i) power line towers along the left bank from km 7.90 to 8.10.

ii) a buried gas pipeline along the right bank from km 9.35 to 10.50.

iii) telephone cable towers along the right bank from km 21.35 to 22.80 and from km 25.50 to 26.00.

The utilities which would be affected by the proposed reservoir and/or regression of the shoreline have been included in Tables 4-1 and 4-2. Estimated volumes of materials required for protection of the utilities have been included in Tables 4-3 and 4-4.

## 4.9 EFFECTS OF MODIFYING THE FULL SUPPLY LEVEL TO 421 m AND 427 m

Although the effects of changing the FSL elevation were not studied in detail, the following section provides a comparison of the general effects of two alternative FSLs, 421 m and 427 m, with the selected 424 m level.

### 421 m FSL

The physical effects of a 421 m FSL along the reservoir shoreline would be reduced from those associated with a 424 m FSL.

Over the entire reservoir shoreline, the Columbia and Kootenay Rivers have experienced water levels in excess of el 421 m and in many areas there is a significant boulder pavement developed up to at least this level.

Inundation associated with a 421 m FSL would be less severe than that associated with a 424 m FSL. Significant areas inundated would be limited to those downstream of Genelle such as portions of the lower residential areas in south China Creek, and recreational land in Birchbank.

Sections of the rail grade south of Genelle would not be inundated. However they would require some reconstruction to comply with minimum freeboard allowances. This would include the bridge structure at Murphy Creek. Accordingly the earthwork quantities required for rail grade reconstruction would be a small fraction of those required for a 424 m FSL.

Although natural slope armouring is present in many areas, some erosion due to beaching may still occur. The natural slopes of a river bank in general tend to flatten from top to bottom. Because of this, many slopes near the 421 m level are presently at or near their long term beach slope. In these cases, little or no shoreline regression would occur, and a safeline would be located at the minimum freeboard elevation. In other areas, where steeper slopes exist, the location of a safeline would not vary significantly from that predicted for a 424 m FSL. Oversteep slopes would remain a problem.

Protection would be required in some areas to prevent shoreline regression from encroaching on existing development. Protection requirements would however be reduced significantly from those associated with a 424 m FSL. Both the total length of protected areas and the volume of protection materials per unit length of protected area would be reduced. The reduction in total length can be reliably determined only by an office study similar in magnitude to that reported for the 424 m FSL. Based on the protected sections selected and summarized in tables 4-3 and 4-4, the volume of protection materials per unit length of protected area for a 421 m FSL would be reduced on average about 28% from those associated with a 424 m FSL.

### 427 m FSL

The physical effects of a 427 m FSL along the reservoir shoreline would be increased from those associated with a 424 m FSL.

Only portions of the reservoir shoreline upstream of km 14 on the Columbia River have experienced water levels to el 427 m or higher, and then only during pre 1963 flood conditions. Consequently, well developed boulder pavement rarely exists above el 424 m.

Inundation would be of larger areal extent than that associated with a 424 m FSL. This is due mainly to the presence of gently sloping river terraces and alluvial fans near the 424 m to 427 m level. Additional areas that would be flooded include portions of the low lying residential area in South Castlegar (km 15.0 to 16.0), the Castlegar sewage lagoon, and sections of highway in the Robson and Pass Creek areas.

With a 427 m FSL, a much longer length of rail grade would be effected, possibly including sections along the right bank upstream of Castlegar. Thus earthwork volumes associated with rail grade reconstruction would be significantly larger than those required for a 424 m FSL.

Although the extent of shoreline subject to beaching would increase, the predicted shoreline regression of those areas previously identified as prone to beaching for the case of a 424 m FSL, would increase only slightly. Thus the location of the safeline in those areas would not vary significantly from that predicted for the 424 m FSL.

With regard to the increased shoreline protection that would be required, based on the protected sections covered in tables 4-3 and 4-4, the volumes of protection materials per unit length of protected area for a 427 m FSL would be increased on average about 35% above those associated with a 424 m FSL. Again however, the increase in the required length of protection would need a detailed office study in order to be reliably determined.

## TABLE 4-1

## SUMMARY OF SAFELINE DISTANCES WITHOUT SHORELINE PROTECTION ALONG THE LEFT BANK

| 0.25 km INTERVALS | SAFELINE OFFSET* (m from FSL) | NOTES LOCATION (km) from - to | REMARKS |
|---|---|---|---|
| 0.00 | - | | - axis of Hugh Keenlyside Dam |
| | | 0.00-0.20 | - ongoing erosion from Keenlyside discharge protection required |
| 0.25 | - | | |
| | | 0.20-0.35 | - bedrock outcrop |
| | | 0.35-0.45 | - erosion from Keenlyside discharge (back eddy) |
| 0.50 | 25 | | |
| 0.75 | 25 | | |
| 1.00 | 40 | | |
| 1.25 | 45 | | |
| | | 1.40-1.60 | - buildings within safeline, safeline at min. freeboard elev. |
| 1.50 | 65 | | |
| 1.75 | 35 | | |
| 2.00 | 35 | | |
| 2.25 | 40 | | |
| 2.50 | 15 | | |
| | | 2.65-2.95 | - buildings within safeline, safeline at min. freeboard elev. |
| 2.75 | 20 | | |
| 3.00 | 20 | | |
| | | 3.10-3.70 | - buildings within safeline, safeline at min. freeboard elev. |
| 3.25 | 20 | | |
| 3.50 | 45 | | |
| 3.75 | 25 | | |
| 4.00 | 20 | | |
| | | 4.10-4.90 | - safeline encroaches on highway |
| 4.25 | 25 | | |
| 4.50 | 25 | | |
| 4.75 | 25 | | |
| 5.00 | 15 | | |
| | | 5.10-5.30 | - safeline encroaches on highway |
| 5.25 | 15 | | |
| | | | - public boat ramp at 5.40 km inundated |
| | | 5.40-5.65 | - safeline encroaches on highway |
| 5.50 | 15 | | |
| | | 5.70-6.55 | - safeline encroaches on highway |
| 5.75 | 15 | | |
| 6.00 | 15 | | |
| 6.25 | 15 | | |
| 6.50 | 15 | | |
| 6.75 | 15 | | |
| | 15 | 6.75-7.20 | - safeline encroaches on highway |
| 7.00 | 15 | | - ferry landing at 7.20 km partially inundated |
| 7.25 | 20 | | |
| | | | - mouth of Pass Creek at 7.35 km |
| Pass Creek - Right Bank | | | |
| 0.00 | 20 | | - mouth of Pass Creek |
| 0.25 | 15 | | |
| 0.50 | 15 | | |
| 0.75 | 5 | | - highway bridge and abutment causeway at 0.75 |
| 1.00 | See fig. C-2 | | - safeline at minimum freeboard elev. |
| 1.25 | See fig. C-2 | | - safeline at minimum freeboard elev. |
| Pass Creek - Left Bank | | | |
| 1.25 | See fig. C-2 | | - safeline at minimum freeboard elev. |
| | | 1.25- 1.20 | - buildings within safeline, safeline at min. freeboard elev. |
| 1.00 | See fig. C-2 | | |
| 0.75 | 5 | | - highway bridge and abutment causeway at 0.75 |
| 0.50 | see fig. C-2 | | |
| 0.25 | see fig. C-2 | | |
| 0.00 | see fig. C-2 | | |

* Refer to Fig. C-2, sheets 1 to 3 for plan summary

TABLE 4-1 (continued)

SUMMARY OF SAFELINE DISTANCES WITHOUT SHORELINE PROTECTION
ALONG THE LEFT BANK

| 0.25 km INTERVALS | SAFELINE OFFSET* (m from FSL) | NOTES LOCATION (km) from - to | REMARKS |
|---|---|---|---|
| 7.50 | see fig. C-2 | | - safeline at min. freeboard elev. |
| 7.75 | see fig. C-2 | | - safeline at min. freeboard elev. |
| | | | - towers for overhead powerlines within safeline from 7.90 to 8.10 km |
| 8.00 | see fig. C-2 | | - at 8.10 km, CPR bridge and towers for overhead powerlines, protection required |
| | | 8.10-8.35 | - safeline threatens sewage lagoon |
| 8.25 | 20 | | |
| | | 8.35-8.95 | - safeline encroaches on railroad grade |
| 8.50 | 45 | | |
| 8.75 | 50 | | |
| | | 8.95-9.30 | - safeline encroaches on existing buildings |
| 9.00 | 35 | | |
| 9.25 | 35 | | |
| | | | - tower for overhead gas pipeline crossing within safeline at 9.35 km |
| | | 9.35-9.70 | - buildings within safeline |
| 9.50 | 45 | | |
| | | 9.70-9.90 | - safeline encroaches on cemetery, protection required |
| 9.75 | 30 | | |
| 10.00 | see fig. C-2 | | |
| | | | - at 10.10 km, towers for overhead telephone cables inundated |
| 10.25 | see fig. C-2 | | |
| | | | - confluence of Columbia and Kootenay Rivers at 10.45 km |
| Kootenay River - Right Bank | | | |
| 0.00 | see fig. C-2 | | - mouth of Kootenay River |
| 0.25 | 35 | | |
| 0.50 | see fig. C-2 | | - safeline at min. freeboard elev. |
| 0.75 | see fig. C-2 | | - safeline at min. freeboard elev. |
| 1.00 | see fig. C-2 | | - safeline at min. freeboard elev. |
| | | 1.00- 1.05 | - buildings within safeline |
| | | 1.15- 1.35 | - buildings within safeline |
| 1.25 | 30 | | |
| 1.50 | 30 | | |
| | | | - towers for overhead powerline crossing at 1.60 km |
| | | | - highway bridge at 1.60 km, safeline at min. freeboard elev. |
| Kootenay River - Left Bank | | | |
| | | | - highway bridge at 1.60 km, safeline at min. freeboard elev. |
| | | | - towers for overhead powerline crossing at 1.55 km within safeline |
| 1.50 | 30 | | - safeline at min. freeboard elev. |
| | | 1.40- 1.15 | drive-in theatre within safeline, partially inundated |
| 1.25 | 75 | | |
| 1.00 | see fig. C-2 | | - safeline at min. freeboard elev. |
| 0.75 | see fig. C-2 | | - safeline encroaches on roadway |
| 0.50 | see fig. C-2 | | |
| 0.25 | see fig. C-2 | | |
| 0.00 | see fig. C-2 | | - mouth of Kootenay River |
| 10.50 | see fig. C-2 | | |
| 10.75 | 35 | | - safeline encroaches building at College, inundated outfall at river level at 10.75 km |
| 11.00 | 35 | | |
| | | | - towers for overhead powerline and telephone cable crossings at 11.20 km |
| 11.25 | 60 | | |
| 11.50 | 60 | | |
| 11.75 | 65 | | |
| | | 11.90-12.20 | - buildings within safeline at crest of slope |
| 12.00 | 50 | | |
| 12.25 | 30 | | |
| 12.50 | 35 | | |
| | | | - towers for overhead powerline crossing at 12.60 km |

* Refer to Fig. C-2, sheets 3 to 5 for plan summary

TABLE 4-1 (continued)

SUMMARY OF SAFELINE DISTANCES WITHOUT SHORELINE PROTECTION
ALONG THE LEFT BANK

| 0.25 km INTERVALS | SAFELINE OFFSET* (m from FSL) | NOTES LOCATION (km) from - to | REMARKS |
|---|---|---|---|
| 12.75 | 100 | | |
| 13.00 | 105 | | |
| | | | - building (possibly pumphouse) inundated at 13.15 km<br>- highway bridge crossing at 13.20 km - protection required |
| 13.25 | 90 | | |
| 13.50 | 40 | | |
| 13.75 | 40 | | |
| 14.00 | 45 | | |
| | | 14.20-14.25 | - buildings within safeline |
| 14.25 | 45 | | |
| 14.50 | 35 | | |
| 14.75 | 25 | | |
| 15.00 | 35 | | |
| | | 15.10-15.30 | - safeline possibly encroaches on cemetery (location of cemetery relative to crest of existing slope uncertain) |
| 15.25 | 40 | | |
| 15.50 | 40 | | |
| | | 15.55-15.85 | - buildings within safeline |
| 15.75 | 35 | | |
| 16.00 | 30 | | |
| | | 16.00-16.30 | - buildings within safeline |
| 16.25 | 25 | | |
| | | | - towers for overhead powerline crossing within safeline at 16.35 km |
| 16.50 | see fig. C-2 | | |
| 16.75 | 5 | | - safeline at min. freeboard elev. |
| 17.00 | 25 | | |
| 17.25 | 35 | | |
| 17.50 | 30 | | |
| 17.75 | 20 | | |
| | | | - overhead utility crossing towers within safeline at 17.90 km |
| 18.00 | 35 | | |
| 18.25 | 35 | | |
| | | | - overhead utility crossing towers within safeline at 18.30 km |
| 18.50 | 35 | | |
| | | 18.65-19.00 | - bedrock exposed at FSL, safeline at min. freeboard elev. |
| 18.75 | 5 | | |
| 19.00 | 5 | | |
| | | 19.00-19.90 | - safeline distance uncertain, existing roadway is within safeline and shoreline will require protection if roadway is to remain |
| 19.25 | - | | |
| 19.50 | - | | |
| 19.75 | - | | |
| | | 19.90-21.00 | - existing roadway inundated |
| 20.00 | 85 | | |
| 20.25 | 40 | | |
| 20.50 | 25 | | |
| 20.75 | 15 | | |
| 21.00 | 25 | | |
| 21.25 | 25 | | |
| 21.50 | see fig. C-2 | | - safeline at min. freeboard elev. |
| 21.75 | 10 | | - safeline at min. freeboard elev. |
| 22.00 | 15 | | |
| | | 22.00-22.30 | - safeline encroaches on cemetery |
| 22.25 | 20 | | |
| 22.50 | 20 | | |
| 22.75 | 25 | | |
| 23.00 | 45 | | |
| 23.25 | see fig. C-2 | | |
| 23.50 | 60 | | |
| 23.75 | 65 | | |
| 24.00 | see fig. C-2 | | |
| 24.25 | 25 | | |
| 24.50 | 45 | | |
| 24.75 | 20 | | |
| 25.00 | 20 | | |
| 25.25 | 25 | | |

*Refer to Fig. C-2, sheets 5 to 9 for plan summary

TABLE 4-1 (continued)

SUMMARY OF SAFELINE DISTANCES WITHOUT SHORELINE PROTECTION ALONG THE LEFT BANK

| 0.25 km INTERVALS | SAFELINE OFFSET* (m from FSL) | NOTES LOCATION (km) from - to | REMARKS |
|---|---|---|---|
| 25.50 | 55 | | |
| 25.75 | see fig. C-2 | | |
| 26.00 | see fig. C-2 | | |
| 26.25 | see fig. C-2 | | |
| 26.50 | see fig. C-2 | | |
| 26.75 | see fig. C-2 | | |
| 27.00 | see fig. C-2 | | |
| 27.25 | 75 | | |
| 27.50 | 60 | | |
| 27.75 | 80 | | |
| 28.00 | 120 | | |
| 28.25 | 150 | | |
| 28.50 | 110 | | |
| 28.75 | 10 | | - safeline at min. freeboard elev. |
| | | | - tower for river metering crossing at 28.90 km |
| 29.00 | 10 | | - safeline at min. freeboard elev. |
| 29.25 | 10 | | - safeline at min. freeboard elev. |
| 29.50 | 10 | | - safeline at min. freeboard elev. |
| 29.75 | 10 | | - safeline at min. freeboard elev. |
| 30.00 | 10 | | - safeline at min. freeboard elev. |
| 30.25 | 10 | | - safeline at min. freeboard elev. |
| 30.50 | 10 | | - safeline at min. freeboard elev. |
| 30.75 | 75 | | |
| 31.00 | 80 | | |
| 31.25 | 65 | | |
| 31.50 | 80 | | |
| | | 31.55-32.45 | - safeline uncertain |
| 31.75 | - | | |
| 32.00 | - | | |
| 32.25 | - | | |
| 32.50 | 55 | | |
| 32.75 | 55 | | |
| 33.00 | 50 | | |
| 33.25 | 55 | | |
| 33.50 | 60 | | |
| | | | - axis of proposed Murphy Creek Dam at approximately 33.65 km |

*Refer to Fig. C-2, sheets 9 and 10 for plan summary

TABLE 4-2

## SUMMARY OF SAFELINE DISTANCES WITHOUT SHORELINE PROTECTION ALONG THE RIGHT BANK

| 0.25 km INTERVALS | SAFELINE OFFSET * (m from FSL) | NOTES LOCATION (km) from - to | REMARKS |
|---|---|---|---|
| 0.00 | - | | - axis of Hugh Keenlyside Dam |
| | - | 0.00-4.45 | - B.C. Timber sawmill and pulp mill - no safeline determined in this area |
| 0.25 | - | | |
| 0.50 | - | | |
| 0.75 | - | | |
| 1.00 | - | | |
| 1.25 | - | | |
| 1.50 | - | | |
| 1.75 | - | | |
| 2.00 | - | | |
| 2.25 | - | | |
| 2.50 | - | | |
| 2.75 | - | | |
| 3.00 | - | | |
| 3.25 | - | | |
| 3.50 | - | | |
| 3.75 | - | | |
| 4.00 | - | | |
| 4.25 | - | | |
| 4.50 | 10 | | |
| 4.75 | 30 | | |
| 5.00 | 15 | | |
| 5.25 | 15 | | |
| 5.50 | 15 | | |
| 5.75 | 20 | | |
| 6.00 | 25 | | |
| 6.25 | 30 | | |
| 6.50 | 25 | | |
| 6.75 | 35 | | |
| 7.00 | 25 | | |
| | | 7.15-7.20 | - buildings within safeline |
| | | 7.20-7.50 | - safeline encroaches on roadway |
| 7.25 | 20 | | - ferry landing at 7.25 km, partially inundated |
| 7.50 | 25 | | |
| | | 7.50-7.60 | - buildings within safeline, safeline encroaches roadway |
| | | 7.60-7.85 | - buildings within safeline |
| 7.75 | 35 | | |
| 8.00 | 30 | | |
| | | | - at 8.10 km, CP Rail bridge and towers for overhead powerline crossing, protection required |
| | | 8.15-8.35 | - safeline encroaches on buildings |
| 8.25 | 50 | | |
| | | 8.35-8.80 | - buildings within safeline |
| 8.50 | 30 | | |
| 8.75 | 30 | | |
| | | 8.80-9.75 | - buildings within safeline, safeline at min. freeboard elev. |
| 9.00 | see fig. C-2 | | - safeline at min. freeboard elev. |
| 9.25 | see fig. C-2 | | - safeline at min. freeboard elev. |
| | | | - towers for overhead gas pipeline crossing at 9.35 km; foundations, and anchors inundated. Buried gas line present within the safeline from this location south to about 10.50 km. See Fig. C-2 for actual areas inundated |
| 9.50 | see fig. C-2 | | - safeline at min. freeboard elev. |
| 9.75 | see fig. C-2 | | - safeline at min. freeboard elev. |
| 10.00 | see fig. C-2 | | - safeline at min. freeboard elev. |
| 10.25 | 20 | | - towers for overhead telephone cable crossing at 10.25 km |
| | | | - island offshore from 10.45 to 10.70 km |
| 10.50 | 20 | | - safeline at min. freeboard elev. |
| 10.75 | 20 | | - safeline at min. freeboard elev. |
| | | 10.80-11.60 | - protection required against reservoir current erosion |
| 11.00 | - | | - towers for overhead telephone cable crossing at 11.00 km |
| | | | - towers for overhead powerline crossing at 11.15 km |
| 11.25 | - | | |
| 11.50 | - | | |

* Refer to Fig. C-2, sheets 1 to 5 for plan summary

TABLE 4-2 (continued)

SUMMARY OF SAFELINE DISTANCES WITHOUT SHORELINE PROTECTION
ALONG THE RIGHT BANK

| 0.25 km INTERVALS | SAFELINE OFFSET * (m from FSL) | NOTES LOCATION (km) from - to | REMARKS |
|---|---|---|---|
| 11.75 | 55 | | - buildings within safeline |
| 12.00 | 55 | | - safeline encroaches on buildings |
| 12.25 | 55 | | - safeline encroaches on buildings |
| 12.50 | 40 | | |
| | | | - CMP outfall into river at 12.55 km |
| 12.75 | 90 | | - buildings within safeline |
| 13.00 | 90 | | - buildings within safeline |
| | | | - highway bridge crossing at 13.15 km, protection required |
| 13.25 | 85 | | |
| | | 13.65-13.70 | - safeline encroaches on rail grade |
| 13.75 | 90 | | |
| | | | - well and/or pump facility inundated at 13.90 km |
| | | 13.90-13.95 | - safeline encroaches on roadway |
| | | 13.95-14.15 | - buildings within safeline |
| 14.00 | 90 | | |
| 14.25 | 80 | | |
| 14.50 | 90 | | |
| | | 14.50-14.55 | - buildings within safeline |
| | | 14.55-14.65 | - safeline encroaches on roadway |
| | | 14.70-14.85 | - buildings within safeline |
| 14.75 | 75 | | |
| | | 14.90-15.00 | - roadway within safeline |
| 15.00 | 80 | | |
| | | 15.05-15.10 | - safeline encroaches on buildings |
| | | 15.10-15.35 | - safeline encroaches on roadway |
| 15.25 | 15 | | |
| | | 15.35-15.60 | - roadway within safeline |
| 15.50 | 10 | | |
| | | 15.60-15.90 | - roadway innundated, safeline encroaches on buildings |
| 15.75 | 15 | | |
| | | 15.95-16.25 | - safeline encroaches on buildings |
| 16.00 | 70 | | |
| 16.25 | 65 | | |
| | | | - overhead powerline crossing at 16.30, towers within safeline |
| 16.50 | 70 | | |
| 16.75 | 55 | | |
| 17.00 | 65 | | |
| 17.25 | 30 | | |
| 17.50 | 40 | | |
| | | 17.60-18.20 | - rail grade within safeline |
| 17.75 | 140 | | |
| 18.00 | 135 | | |
| 18.25 | 90 | | |
| | | 18.25-18.30 | - overhead powerline crossings, towers within safeline |
| 18.50 | 80 | | |
| | | 18.55-19.10 | - safeline encroaches on buildings |
| 18.75 | 75 | | |
| 19.00 | 75 | | |
| 19.25 | 75 | | |
| 19.50 | 80 | | |
| 19.75 | 85 | | |
| | | 19.75-19.80 | - safeline encroaches on buildings |
| 20.00 | see fig. C-2 | | |
| 20.25 | 80 | | |
| 20.50 | 60 | | |
| 20.75 | 80 | | |
| 21.00 | 80 | | |

* Refer to Fig. C-2, sheets 5 to 8 for plan summary

TABLE 4-2 (continued)

SUMMARY OF SAFELINE DISTANCES WITHOUT SHORELINE PROTECTION
ALONG THE RIGHT BANK

| 0.25 km INTERVALS | SAFELINE OFFSET * (m from FSL) | NOTES LOCATION (km) from - to | REMARKS |
|---|---|---|---|
| 21.25 | 70 | | |
| | | 21.35-21.40 | - safeline encroaches on rail grade<br>- towers for overhead telephone cable inundated and/or threatened from 21.40 to 22.80 km |
| 21.50 | 80 | | |
| 21.75 | 75 | | |
| 22.00 | 80 | | |
| 22.25 | 85 | | |
| | | 22.30-22.55 | - safeline encroaches on rail grade |
| 22.50 | 85 | | |
| 22.75 | 95 | | |
| | | 22.80-23.85 | - buildings inundated and within safeline |
| 23.00 | 25 | | |
| 23.25 | 20 | | |
| 23.50 | 20 | | |
| 23.75 | 20 | | |
| | | 23.85-24.50 | - buildings innundated, safeline encroaches on rail grade |
| 24.00 | 30 | | |
| 24.25 | 30 | | |
| 24.50 | 25 | | |
| 24.75 | 20 | | |
| 25.00 | 20 | | |
| 25.25 | see fig. C-2 | | - safeline at min. freeboard elev. through gravel pit |
| 25.50 | 10 | | |
| | | | - towers for overhead telephone cable inundated and/or threatened from 25.50 to 26.00 km |
| 25.75 | 20 | | |
| 26.00 | 20 | | |
| | | 26.20-27.95 | - safeline encroaches on rail grade, except for the area from 27.50 to 27.60 where the rail grade is in a bedrock cut and safeline is at min. freeboard elev. |
| 26.25 | 15 | | |
| 26.50 | 30 | | |
| 26.75 | 25 | | |
| 27.00 | 20 | | |
| 27.25 | 15 | | |
| 27.50 | 5 | | |
| 27.75 | 15 | | |
| | | 27.95-28.25 | - rail grade within safeline |
| 28.00 | 30 | | |
| 28.25 | 20 | | |
| | | 28.25-32.95 | - rail grade inundated |
| 28.50 | 30 | | |
| 28.75 | 40 | | |
| | | | - tower for river metering crossing at 28.90 km inundated |
| 29.00 | 55 | | |
| 29.25 | 70 | | |
| 29.50 | 60 | | |
| 29.75 | 5 | | - safeline at min. freeboard elev. bedrock exposure extending 10 m north and 40 m south of 29.75 km |
| 30.00 | 60 | | |
| 30.25 | 55 | | |
| 30.50 | 60 | | |
| 30.75 | 50 | | |
| 31.00 | 60 | | |
| 31.25 | 65 | | |
| 31.50 | 60 | | |
| 31.75 | see fig. C-2 | | |
| | | | - rail bridge inundated at 31.80 km |
| 32.00 | 25 | | |
| 32.25 | 25 | | |
| 32.50 | 25 | | |
| 32.75 | 5 | | - safeline at min. freeboard elev. |
| 33.00 | 5 | | - safeline at min. freeboard elev. |
| 33.25 | 5 | | - safeline at min. freeboard elev. |
| 33.50 | 5 | | - safeline at min. freeboard elev. |
| | | | - axis of proposed Murphy Creek Dam at about 33.65 km. |

*Refer to Fig. C-2, sheet 8 to 10 for plan summary

TABLE 4-3

## QUANTITY SUMMARY FOR SHORELINE PROTECTION FOR SELECTED AREAS ALONG THE LEFT BANK

| LOCATION OF PROTECTED AREA (km) from - to | VOLUME OF PROTECTION MATERIAL |  |  | REMARKS |
|---|---|---|---|---|
|  | Typ. Section area (m$^2$) | Length (m) | Volume (m$^3$) |  |
| 0.00 - 0.20 | 24 | 320 | 7680 | - protection for dam tailrace erosion |
| 0.35 - 0.45 | 14 | 120 | 1680 |  |
| 4.10 - 4.90 | 19 | 860 | 16340 | - protection for highway |
| 5.10 - 5.30 | 40 | 150 | 6000 | - protection for highway |
| 5.40 - 5.65 | 40 | 230 | 9200 | - protection for highway |
| 5.70 - 6.55 | 40 | 820 | 3280 | - protection for highway |
| 6.75 - 6.85 | 5 | 110 | 550 | - protection for highway |
| 6.85 - 7.20 | 19 | 290 | 5510 | - protection for highway |
| 8.05 - 8.15 | 39 | 100 | 3900 | - protection for CP Rail Bridge, and powerline towers |
| 8.15 - 8.25 | 30 | 100 | 3000 | - protection for sewage lagoon |
| 8.25 - 8.35 | 18 | 500 | 9000 | - protection for sewage lagoon |
| 8.35 - 8.95 | 18 | 780 | 14040 | - protection for rail grade |
| 8.95 - 9.30 | 21 | 490 | 10290 | - protection for buildings |
| 9.30 - 9.35 | 30 | 70 | 2100 | - protection for overhead gas pipeline tower, slope may require reconstruction |
| 9.35 - 9.70 | 30 | 430 | 12900 | - protection for buildings |
| 9.70 - 9.90 | 30 | 200 | 6000 | - protection for cemetary, slope may require reconstruction |
| Kootenay River - left bank |  |  |  |  |
| K1.60 - 1.55 | 12 | 40 | 480 | - protection for overhead powerline tower |
| 10.70 - 10.80 | 15 | 90 | 1350 | - protection for College building |
| 11.10 - 11.30 | 15 | 180 | 2700 | - protection for overhead powerline and telephone cable towers |
| 14.15 - 14.30 | 14 | 150 | 2100 | - protection for buildings |
| 15.00 - 15.40 | 39 | 430 | 16770 | - protection for cemetary |
| 15.50 - 15.95 | 36 | 450 | 16200 | - protection for buildings |
| 16.00 - 16.30 | 35 | 350 | 12250 | - protection for building |
| 16.30 - 16.40 | 21 | 100 | 2100 | - protection for overhead powerline tower |
| 17.85 - 17.95 | 13 | 90 | 1170 | - protection for overhead powerline tower |
| 18.25 - 18.35 | 14 | 80 | 1120 | - protection for overhead powerline tower |
| 18.95 - 19.90 | 12 | 1040 | 12480 | - protection for roadway |
| 19.90 - 21.00 | 17 | 1170 | 19890 | - raise and protect roadway |
| 21.90 - 22.40 | 14 | 490 | 6860 | - protection for cemetary |

Total length of shoreline protection for selected areas along left bank = 10.23 km
Total volume of protection material for selected areas along left bank = 206940 m$^3$

TABLE 4-4

## QUANTITY SUMMARY FOR SHORELINE PROTECTION FOR SELECTED AREAS ALONG THE RIGHT BANK

| LOCATION OF PROTECTED AREA (km) from - to | VOLUME OF PROTECTION MATERIAL | | | REMARKS |
|---|---|---|---|---|
| | Typ. Section area ($m^2$) | Length (m) | Volume ($m^3$) | |
| 7.20 - 7.50 | 12 | 290 | 3480 | - protection for highway |
| 7.50 - 7.85 | 12 | 370 | 4440 | - protection for buildings |
| 8.05 - 8.15 | 16 | 100 | 1600 | - protection for CP Rail Bridge |
| 8.15 - 8.35 | 16 | 210 | 3360 | - protection for buildings |
| 8.35 - 8.45 | 16 | 100 | 1600 | - protection for buildings, slope may require reconstruction. |
| 8.45 - 8.70 | 16 | 230 | 3680 | - protection for buildings |
| - - | 6 | 300 | 1800 | - protection for island offshore from 10.45 to 10.70 km |
| 10.80 - 11.10 | 12 | 310 | 3720 | - protection against current erosion |
| 11.10 - 11.60 | 21 | 500 | 10500 | - protection against current erosion |
| 11.60 - 12.65 | 21 | 930 | 19530 | - protection for buildings |
| 12.65 - 12.75 | 21 | 130 | 2730 | - protection against current erosion |
| 12.75 - 13.15 | 27 | 370 | 9990 | - protection against current erosion |
| 13.15 - 13.50 | 27 | 330 | 8910 | - protection against current erosion |
| 13.50 - 13.75 | 20 | 320 | 6400 | - protection for rail grade |
| 13.85 - 13.95 | 14 | 120 | 1680 | - protection for roadway |
| 13.95 - 14.20 | 14 | 250 | 3500 | - protection for buildings |
| 14.45 - 15.00 | 12 | 550 | 6600 | - protection for buildings and roadway |
| 15.00 - 15.90 | 33 | 890 | 29370 | - protection for roadway and buildings, estimated quantity does not provide for possible reconstruction of road to raise to min. freeboard elev. |
| 15.95 - 16.25 | 14 | 320 | 4480 | - protection for buildings |
| 16.25 - 16.30 | 14 | 50 | 700 | - protection for towers of overhead powerline crossing |
| 17.55 - 18.25 | 14 | 860 | 12040 | - protection for rail grade |
| 18.25 - 18.35 | 15 | 150 | 2250 | - protection for towers of overhead powerline crossing |
| 18.50 - 19.15 | 15 | 630 | 9450 | - protection for buildings |
| 19.70 - 19.85 | 25 | 130 | 3250 | - protection for buildings |
| 21.30 - 21.45 | 13 | 120 | 1560 | - protection for rail grade, slope may require reconstruction |
| 22.25 - 22.60 | 29 | 440 | 12760 | - protection for rail grade |
| 23.85 - 24.50 | 14 | 300 | 4200 | - protection for rail grade |
| 26.15 - 26.75 | 82 | 740 | 60680 | - protection for rail grade |
| 26.75 - 28.25 | 32 | 1450 | 46400 | protection for rail grade |
| 28.25 - 32.95 | - | - | | - CP Rail grade to be realigned; protection requirements will depend on new alignment |

Total length of shoreline protection for selected areas along right bank = 11.49 km
Total volume of protection material for selected areas along right bank = 280660 $m^3$

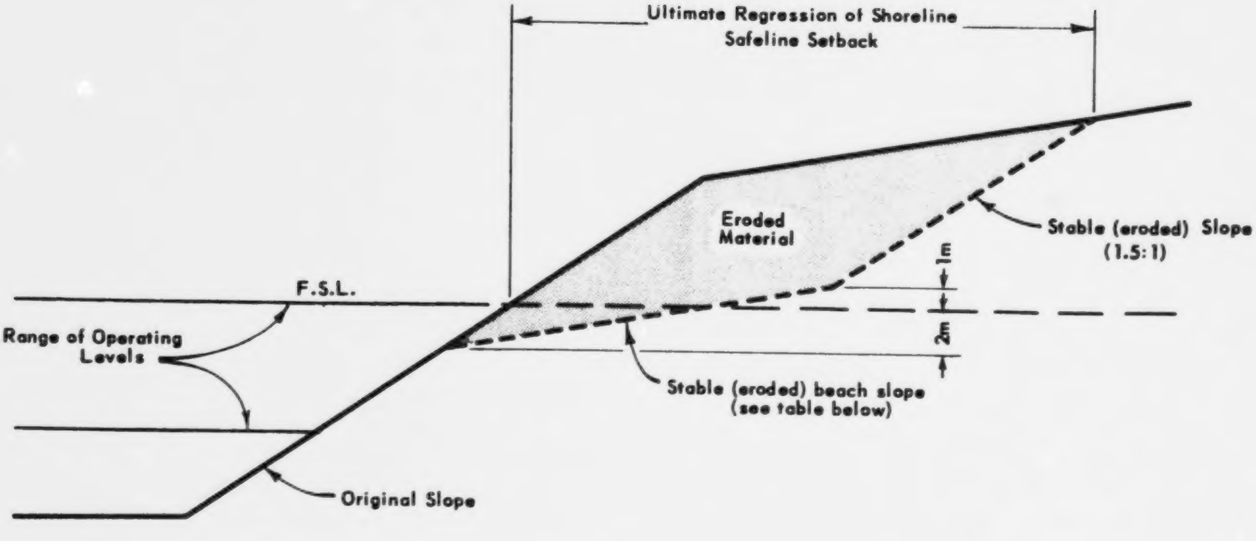
## TABLE 4-5

## UTILITY CROSSINGS OF COLUMBIA AND KOOTENAY RIVERS WITHIN THE PROPOSED RESERVOIR AREA*1

| Type of Utility | Location of Crossing River Centreline (km) | Elevation of Tower Base (m) | | Remarks |
|---|---|---|---|---|
| | | Left Bank | Right Bank | |
| Columbia River | | | | |
| Power | 1.00 | 490.7 | 442.1 | - tower upslope of safeline*2 |
| Power | 8.10 | 425.6 | 443.9 | - within safeline, protection required<br>- tower upslope of safeline |
| Gas Pipeline | 9.35 | 437.0 | 422.0 | - within safeline, protection and/or slope reconstruction required<br>- Tower foundation inundated |
| Telephone | 10.15 | 425.0 | 425.5 | - area surrounding tower inundated<br>- tower below minimum freeboard elev. |
| Telephone | 11.05 | 454.2 | 433.3 | - within safeline, protection required<br>- within safeline, protection required |
| Power | 11.15 | 459.2 | 452.5 | - tower upslope of safeline<br>- tower upslope of safeline |
| Power/Telephone | 12.50 | 464.3 | 447.3 | - tower is upslope of safeline<br>- within safeline, protection required |
| Power | 16.35 | 428.1 | 455.0 | - within safeline, protection reuqired<br>- within safeline, protection required |
| Power | 18.10 | 427.0 | 476.9 | - within safeline, protection required<br>- within safeline, protection required |
| Power | 18.30 | 426.8 | 474.2 | - within safeline, protection required<br>- within safeline, protection required |
| River Metering Cable | 28.90 | 428.5 | 420.4 | - tower upslope of safeline<br>- tower foundation inundated |
| Kootenay River | | | | |
| Power | 1.60 | 425. | 436. | - within safeline, protection required<br>- tower upslope of safeline |
| Power | 1.65 | 480. | 458. | - safelines not determined in this area<br>- safelines not determined in this area |

Notes: 1. List of crossings compiled on basis of B.C. Hydro survey information received March 24, 1982. List does not include Bridge Crossings

2. "Safeline" referred to in table is that for the case where no protection is provided.


Ultimate Regression of Shoreline
Safeline Setback
Eroded Material
Stable (eroded) Slope (1.5:1)
1m
F.S.L.
2m
Range of Operating Levels
Stable (eroded) beach slope (see table below)
Original Slope

| [1]*Beach Slope (H:V) | Material [2]* | |
|---|---|---|
| 10:1 | fine sand | ($D_{50}$ = 0.2 to 0.4mm) |
| 8:1 | medium sand | ($D_{50}$ = 0.4 to 2.0mm) |
| 7:1 | coarse sand | ($D_{50}$ = 2.0 to 5.0mm) |
| 6:1 | fine gravel | ($D_{50}$ = 5.0 to 20mm) |
| 5:1 | coarse gravel | ($D_{50}$ = 20 to 75mm) |
| 4:1 | cobbles | ($D_{50}$ = 75 to 125mm) |
| 3:1 | cobbles | ($D_{50}$ = 125 to 200mm) |

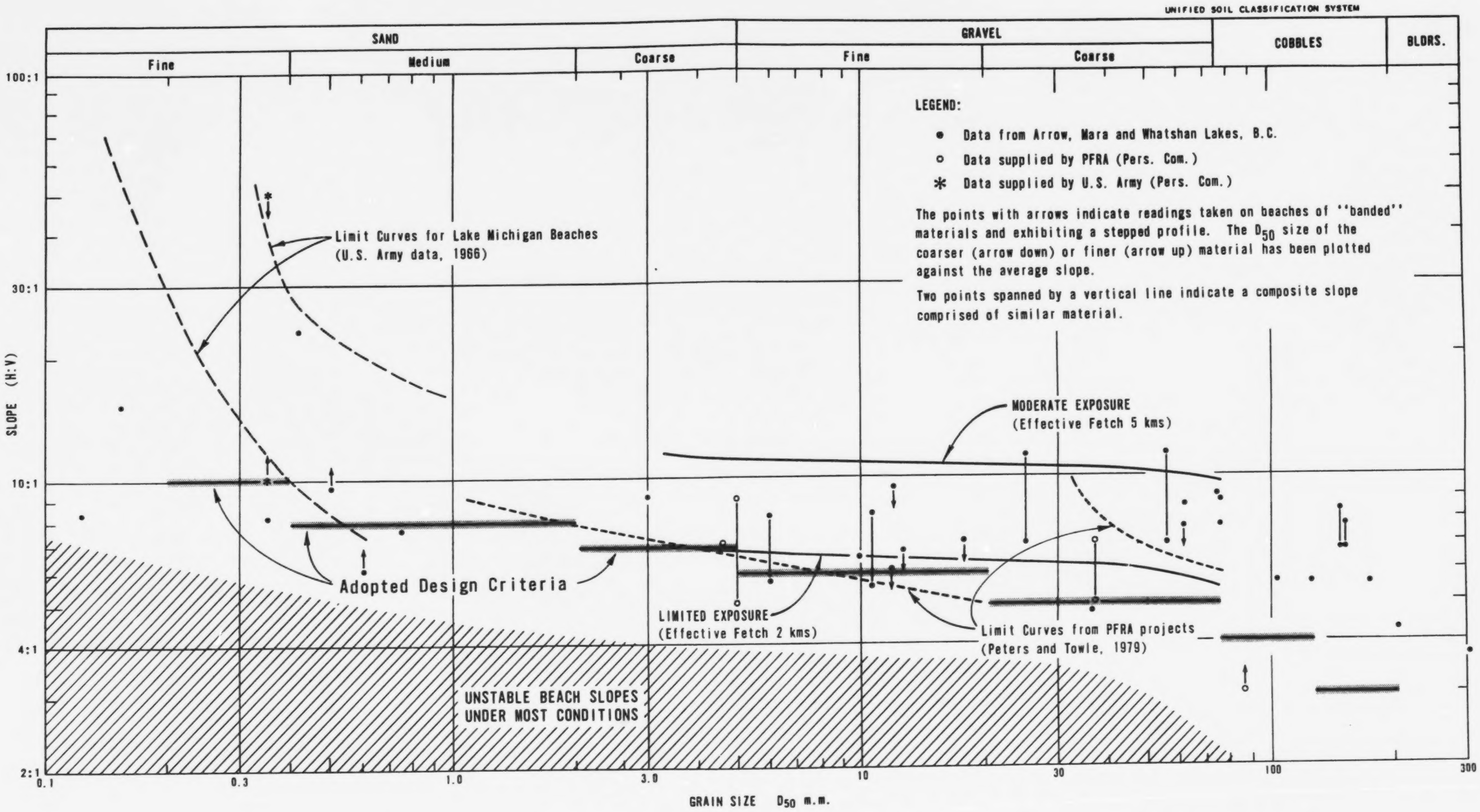
*NOTES:

1. Beach slopes taken from Figure 4-2
2. Terminology corresponds to Unified Soil Classification System.
3. It is assumed that the eroded material does not contribute to the beach slope.

ELEMENTS OF BANK EROSION BY BEACHING

THURBER

FIGURE 4-1



● Data from Arrow, Mara and Whatshan Lakes, B.C.

○ Data supplied by PFRA (Pers. Com.)

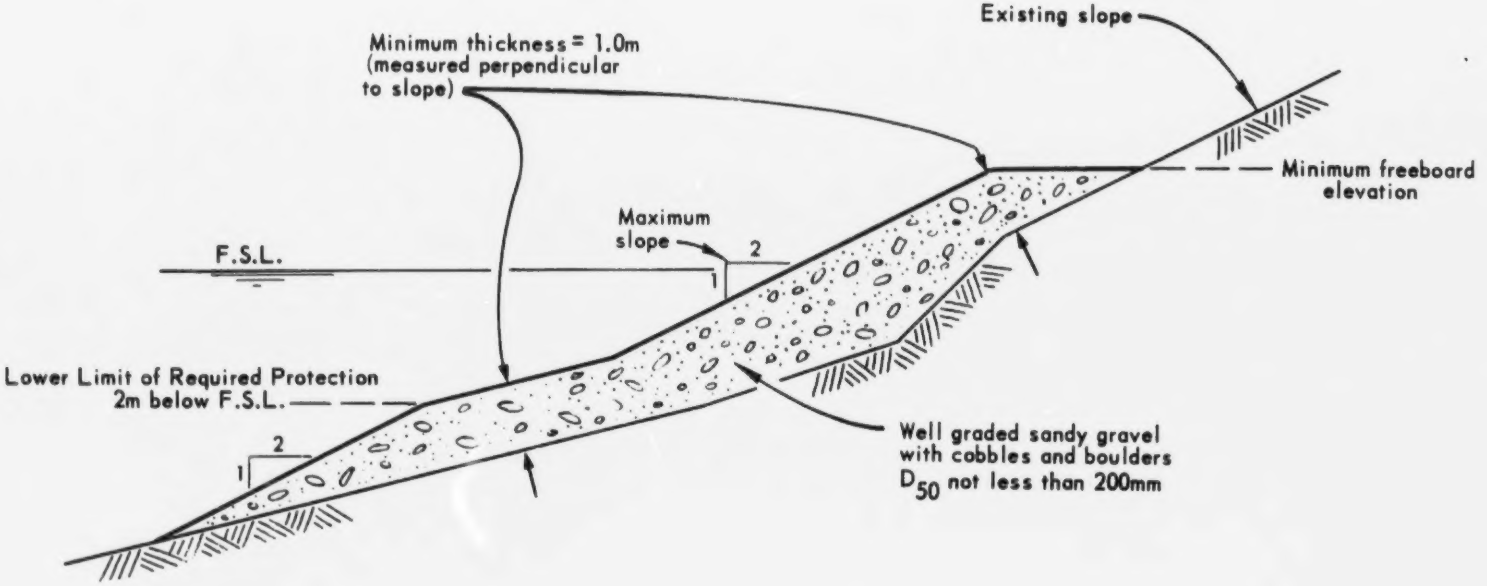
* Data supplied by U.S. Army (Pers. Com.)

The points with arrows indicate readings taken on beaches of ''banded'' materials and exhibiting a stepped profile. The $D_{50}$ size of the coarser (arrow down) or finer (arrow up) material has been plotted against the average slope.

Two points spanned by a vertical line indicate a composite slope comprised of similar material.

GRAIN SIZE ($D_{50}$) VERSUS BEACH SLOPES FOR INLAND LAKES AND RESERVOIRS

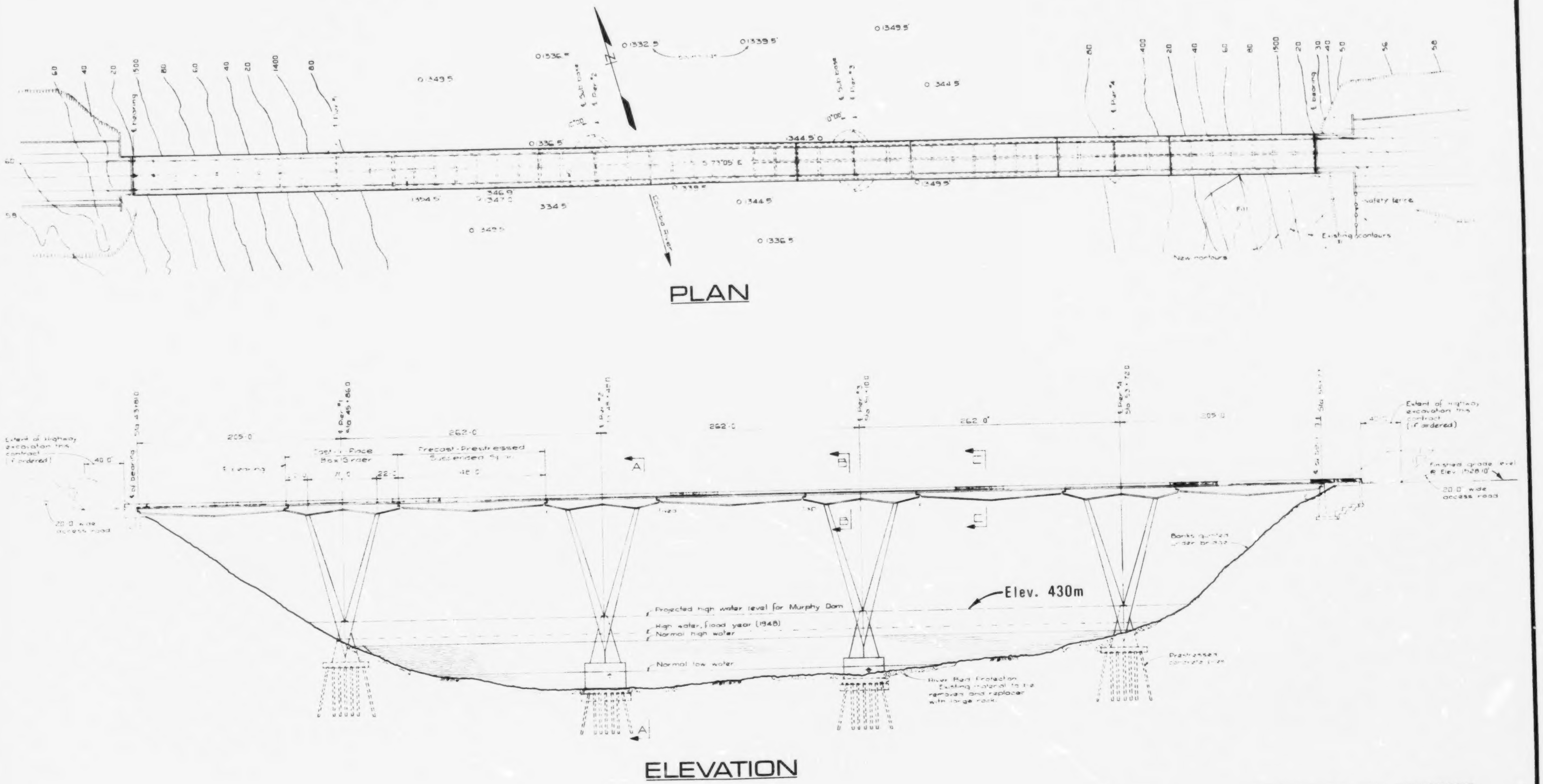
FIGURE 4-2


Existing slope
Minimum thickness = 1.0m (measured perpendicular to slope)
Minimum freeboard elevation
Maximum slope
2
1
F.S.L.
Lower Limit of Required Protection 2m below F.S.L.
2
1
Well graded sandy gravel with cobbles and boulders $D_{50}$ not less than 200mm

NOTE: Maximum 2:1 slope corresponds to material noted.

CONCEPTUAL DESIGN
FOR
PROTECTIVE WORKS

FIGURE 4-3


PLAN
ELEVATION
N
Columbia River
S 73°05' E
Sub base
Pier #1
Pier #2
Pier #3
Pier #4
bearing
Safety fence
Fill
Existing contours
New contours
205.0
262.0
262.0
262.0
205.0
Cast-in-Place Box Girder
Precast-Prestressed Suspended Span
Extent of highway excavation this contract (if ordered)
40.0
20.0' wide access road
Finished grade level
Elev. 1528.10'
Banks grouted under bridge
Elev. 430m
Projected high water level for Murphy Dam
High water, flood year (1948)
Normal high water
Normal low water
River Bed Protection: Existing material to be removed and replaced with large rock
Prestressed concrete piles
A
B
C

DETAILS OF THE EXISTING
HIGHWAY BRIDGE CROSSING
OF THE COLUMBIA RIVER

**NOTE:** Drawing taken from Gov't. of the Prov. of British Columbia, Dept. of Highways drawing no. 1656-4x dated February 10, 1968 (as built).

FIGURE 4-4

## Section 5

## SUMMARY

The shoreline assessment provided in this report is based primarily on field work carried out during 1981, as outlined in Section 1.4 of this report.

### 5.1 EXISTING CONDITIONS

a) The Columbia River in the proposed reservoir area flows within steep incised banks consisting generally of non-cohesive granular silts, sands and gravels or mixtures thereof. The soils upstream of Castlegar are generally finer than those downstream. No cohesive materials were encountered in proximity to the proposed reservoir.

b) The present river channel is essentially stable under present regulated conditions with adequate natural armouring extending 1 to 2 m above normal high water levels.

c) Several sections of the river (particularly along the right bank just south of the Columbia - Kootenay River confluence extending to South Castlegar) have become oversteep by river erosion during previous high water conditions. Many of these slopes are subject to sloughing or ravelling and presently pose a safety hazard to some facilities near their crest.

d) No evidence of recent large scale sliding or slumping is apparent in the proposed reservoir area.

e) The information obtained to date from the piezometer installations indicates the gradient of the groundwater surface is fairly flat - possibly 2 - 3% in the Castlegar area.

f) The Columbia River downstream of the Keenleyside dam is traversed by the CPR at Castlegar and Highway 3 at the south end of the Airport. In addition, there are

13 separate utility or cable crossings: a gas pipeline crossing at km 9.35; powerline crossings at km 1.00, 8.10, 11.15, 12.50, 16.35, 18.10 and 18.30 of the Columbia River and at km 1.60 and 1.65 of the Kootenay River; telephone cable crossings at km 10.15 and 11.05; and a metering cable crossing at km 28.90. With the exception of the CPR bridge which has experienced scour around the pier foundations, these crossings appear to have secure foundations.

g) The rail grade within the proposed reservoir area is at present, generally stable. It is apparent that a 1.5 to 2.0 km section of the railway immediately north of Genelle was previously relocated (at the turn of the century) to avoid a steep sloughing bank.

## 5.2 CONDITIONS AFTER RESERVOIR FILLING

a) This study is primarily concerned with the effects of a reservoir filled to a FSL of 424 m. With this reservoir level, 34 km of the Columbia River extending downstream of the Hugh Keenleyside dam would be inundated. However, prior to 1963, the upstream 20 to 25 km of the river's banks experienced flood levels up to and exceeding the maximum normal operating level for the proposed reservoir.

b) The reservoir would be narrow with a small storage capacity, thus the flow through the reservoir would result in a slight gradient of the water surface (Fig. 3-1). The velocity through the reservoir would be significant (Table 3-1) although reduced from the velocities presently and previously experienced.

c) The predominant wind directions within the reservoir area parallel the valley axes. The results of the study based on a randomly selected year (1967) indicate that winds capable of generating wave heights of 0.25 m or more blow for 11 percent of the year, and that the significant wave height (using a design wind speed of 120 km/hr and an average fetch of 800 m) would be 0.7 m. Much of the wave generating wind activity would occur when the reservoir level is within 1 m or so of FSL.

d) Because of the nature of the soils surrounding the reservoir area, the risk of large scale sliding or slumping due to filling or drawing down of the reservoir is considered to be very low. However without protection, shoreline regression would result in some areas from wave erosion (beaching). Further, the right bank from about km 10.8 to 11.6 is not sufficiently armoured to withstand flows through the reservoir during flood conditions and significant regression due to reservoir current erosion could result. Of the 85 km of reservoir shoreline, 58 km would be prone to shoreline regression. The areas that could be impacted include: residential property in North and South Castlegar, Blueberry, and China Creek; cemetaries in Brilliant, Ootischenia and the Champion Creek area; road grades at Robson; and railway grades near China Creek and Blueberry.

e) Beaching would be most active over a 3 m range, namely FSL + 1 m and -2 m. The resulting shoreline regression would normally occur slowly, taking many years to reach its predicted ultimate limit.

f) Shoreline regression resulting from wave and current erosion of normal vegetated slopes (not oversteep slopes) can be prevented by construction of a protective layer of suitably coarse granular material containing cobbles of average 200 mm diameter (Fig. 4-3). Sources of potentially suitable material exist in the fans of Blueberry, China and Murphy Creeks. Quantity estimates of minimum protection requirements have been computed (Tables 4-3 and 4-4) to sufficient accuracy to permit an assessment of the economic viability of bank protection. Riprap sources could be made available from proposed rock excavations at the Hugh Keenleyside and Murphy Creek damsites, however the suitability of this material (gradation) and possible variation to the configuration of the protective layer shown in Fig. 4.3 has not been considered. Protection of 22 km of shoreline has been considered.

g) Regression of an oversteep slope can only be prevented by reconstruction of the slope to a stable configuration, and would not be halted by construction of a protecive layer. Areas where the slopes are oversteep have been identified and are shown on the drawings in Appendix C. In some of these areas houses are located within a few metres of the crest of the slopes.

h) Of the 36 km of CPR trackage within the reservoir area, 4.7 km would be inundated and an additional 4.9 km of the grade would be threatened by regression of the shoreline. In addition to the rail grade, the bridge structure across the Columbia River may be affected, and the bridge across Murphy Creek would be inundated. The effect of the reservoir on the railway is the subject of a separate study by Thurber Consultants.

i) Design drawings for the highway bridge crossing of the Columbia River indicate a Murphy Creek Project with a FSL of 430 m was taken into account in the design. Some protection may be required however, at the bridge abutments.

j) Ten of the existing 13 utility crossings of the Columbia River may be affected either by inundation of the tower footings or by shoreline regression unless protected. Details are provided in Section 4.7 and Table 4-5. In addition some utilities following close to the existing river banks could be affected by shoreline regression.

k) Effect of reservoir flooding on the B.C. Timber Pulpmill and Sawmill (km 0.0 to 4.5 along right bank) is the subject of a separate study.

## 5.3 SAFELINE

a) A safeline is a conservatively located line beyond which the security of residents and residential improvements can be reasonably assured. Where no shoreline regression was anticipated the safeline was established at the minimum freeboard elevation.

b) Groundwater considerations such as potential flooding of basements and utilities were not taken into account in locating the safeline. A separate study of this aspect is underway by B.C. Hydro.

c) The preliminary safelines provided in this report have been established along the reservoir assuming no protection. Typical set-backs from the FSL in residential areas are provided below.

| Location | Shoreline Section | Safeline Setback |
|---|---|---|
| N. Castlegar | km 7.25 to 9.00 | 20 to 50 m |
| | 9.00 to 10.10 | 50 to 130 m |
| | 10.10 to 10.80 | 15 to 20 m |
| | 10.80 to 13.15 | 40 to 90 m |
| S. Castlegar | km 13.15 to 15.00 | 75 to 95 m |
| | 15.00 to 15.90 | 5 to 30 m |
| | 15.90 to 16.50 | 50 to 70 m |
| Blueberry | km 18.40 to 19.80 | 75 to 90 m |
| China Creek | km 22.70 to 22.80 | 90 to 100 m |
| | 22.80 to 24.50 | 20 to 30 m |

Details are provided in Tables 4-1 and 4-2 and on 1:10000 scale plans in Appendix C. Also, working drawings are available at a scale of 1:5000 (not part of this report).

d) A summary of the areas of waterfront land falling below the safeline down to the present normal high river level (profile 1, Fig. 3-1) is given below:

| Location | Shoreline Section | Area below Safeline |
|---|---|---|
| Pass Creek | km 7.40 to 8.10 | 45 ha |
| North Castlegar | km 8.80 to 10.30 | 25 ha |
| Brilliant | km K0.00 to K1.15 | 25 ha |
| Selkirk College | km K0.40 to K1.50 | 35 ha |
| China Creek | km 22.70 to 24.50 | 25 ha |
| Birchbank | km 28.50 to 31.00 | 60 ha |

## 5.4 EFFECTS OF MODIFYING THE FSL TO 421 AND 427 m

a) The study included a comparison of the general effects of two alternative FSLs, 421 m and 427 m with the selected 424 m level.

b) Changing the FSL to 421 m would result in:

- less indundation of developed areas, the most significant areas being limited to those downstream of Genelle.
- elimination of flooding of CPR tracks, although some raising of the tracks south of Genelle and the Murphy Creek bridge would still be needed to comply with minimum freeboard allowances.
- reduction in the extent of shoreline subject to beaching and thus the total area of land falling below the safeline.

- reduction in the total quantities required for protection of at least 28%.

c) Changing the FSL to 427 m would result in:

- more inundation of developed areas, including areas in South Castlegar, the Castlegar sewage lagoon, and sections of highway in the Robson and Pass Creek areas.
- the length of the CPR grade affected by flooding and erosion would be increased, and may include the right bank upstream of Castlegar.
- increase in the extent of shoreline subject to beaching, although the safeline location would not significantly change in those areas identified as prone to beaching for the FSL 424 m case.
- increase in the total quantities required for protection of at least 35%.

# Section 6

## RECOMMENDATIONS

a) One of the purposes of this study has been to establish a preliminary safeline for the reservoir as an aid in deciding on where shoreline protection can be economically justified. In areas where protection is not to be provided and where a detailed safeline location is required, site specific work is recommended. This includes detailed topographic mapping to a scale of 1:1250, specific computation of fetch and run-up as it affects beaching and freeboard requirements, and a detailed check of the bank materials.

b) A conceptual design for the protective works has been prepared and utilized in this study. Detailed design and costing would also require further site specific considerations. These include detailed topographic mapping of the pertinent shoreline areas to a scale of 1:1250 supplemented by ground surveyed sections, logistics and construction procedures, detailing of slope conditions (particularly vegetation and any evidence of oversteepening), and nature of construction materials. Consideration should also be given to the viability of scheduling protective works in some areas immediately after filling, thus taking advantage of better water access.

c) Detailed field and laboratory investigations of construction materials to be used in protective works is required prior to proceeding with detailed design and costing.

d) The 424 m FSL, should be established in the field and the findings of this report together with any proposed protection design measures should be rechecked on site.

e) Monitoring of the piezometers installed to date should continue. It is recommended that the piezometers be read monthly during April to November and with increased frequency during periods of high precipitation and/or rapid changes in river level.

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# APPENDIX A

# FIELD INVESTIGATION

A.1 Test Hole Drilling Program
A.2 Test Pit Program
A.3 Slope Mapping Program
A.4 Reconnaissance Visits
A.5 Laboratory Testing
A.6 Other

## A.1 TEST HOLE DRILLING PROGRAM

From June 9 to July 9, 1981, a total of 39 test holes, varying in depth to up to 62 m, were drilled adjacent to the proposed reservoir area using a truck-mounted Becker hammer drill.

The soil was logged during the drilling progress and disturbed bag samples and undisturbed Shelby tube samples (where appropriate) were collected for subsequent laboratory testing. As well, Standard Penetration Tests were carried out at frequent intervals in each of the test holes.

Following completion of drilling, standpipe piezometers were installed in each of the test holes. The piezometers consisted of 25 mm diameter solid PVC pipe with a 1.5 m length of slotted PVC pipe at the lower end. Cast iron caps were fitted over each of the piezometer installations for protection.

The elevation and location of each of the test holes were determined by B.C. Hydro's survey crew from Nelson, B.C.

The location of the test holes are shown on Fig. A-3 (sheets 1 through 10). A summary of the soil stratigraphy encountered in the test holes and the ground surface elevation at each of the test hole locations is presented on Fig. A-1 (sheets 1 through 5). Pertinent elevations concerning the piezometer installation are summarized in Table A-3.

## A.2 TEST PIT PROGRAM

On July 6, 1981, a total of 6 test pits were excavated in the reservoir area using a rubber tired backhoe. The soil stratigraphy was logged during excavation, and disturbed samples were collected for laboratory testing.

The location and elevation of each of the test pits (as determined by B.C. Hydro) are provided on Fig. A-3 (sheets 3 and 5) and Fig. A-2 respectively. The subsurface information collected from the test pits is summarized on the appropriate logs on Fig. A-2.

## A.3 SLOPE MAPPING PROGRAM

The slope mapping program consisted of examining soil exposures on existing slopes around the reservoir and recording information which would be pertinent to the safeline study. Because of the river water level fluctuations, the program was carried out in two stages.

Stage I was carried out from June 14 to July 11, 1981, when river levels were high. During this time, approximately 90 slopes were mapped around the reservoir wherever slope exposures were apparent and access was possible by road with vehicles and/or on foot and by water with small aluminum boats. The soil stratigraphy exposed on the slopes was logged and sampled, the slopes were measured and photographed, and observations concerning proximate structures etc. were recorded.

Stage II was carried out between October 15 to 19 and November 4 to 7, 1981 (two periods). The information collected during this stage was similar to that obtained during Stage I. This stage was concentrated primarily in the built up areas around the reservoir, its purpose being primarily to contribute to and provide information in between those sections mapped during the first stage. The mapping for Stage II was carried out by walking the shoreline of the existing river, which was possible at this time because of low river water levels.

In total, 216 slopes were mapped. The locations of the sections, which were determined from aerial photographs are shown on Fig. A-3 (sheets 1 through 10). Pertinent information extracted from the program is summarized in Tables A-1 and A-2 for the left and right banks respectively. Further details are available in our files.

## A.4 RECONNAISSANCE VISITS

The reconnaissance for this project was carried out in several stages throughout the period of the field program, and for the most part, in conjunction with engineering supervision of the field program.

During April 29, 30 and May 1, 1981 an initial site visit was made to confirm the extent and location of required field testing, and as well obtain permission to enter private properties. The latter was quite adequately accomplished by B.C. Hydro's representative in Castlegar.

On June 8, 1981, a second site visit was carried out to initiate the field program. During the visit (duration of approximately two weeks) a visual survey of the entire reservoir shoreline was carried out by helicopter and by boat (by boat since high water levels prevented hiking around the reservoir shoreline). Subsequent to this, continuous photographs of the reservoir shoreline were taken from a helicopter, providing a visual record and a valuable aid for later work.

A final reconnaissance was carried out during Stage II of the slope mapping program at which time the entire reservoir shoreline was walked with exception of the left bank from the south end of Ootischenia downstream to the proposed Murphy Creek Dam site. The purpose of this final reconnaissance was to piece together information obtained to that time and to identify and/or investigate items of uncertainty.

## A.5 LABORATORY TESTING

A total of 535 disturbed bag samples and 10 undisturbed Shelby tube samples were collected during the field program.

Laboratory testing included visual classification of all samples received. In addition, grain size distribution tests were carried out on selected samples. The laboratory test results have been incorporated into the test hole and test pit logs on Fig. A-1 and A-2 respectively.

## A.6 OTHER

In addition to the field and laboratory testing programs noted above, collection of other site data was undertaken including a hydrological study by Kellerhals Engineering Services Ltd. Also, survey sectioning was carried out by B.C. Hydro surveyors.

Examination of other site data included a review of existing water well logs (provided by Water Rights Branch, Department of Lands, Forests and Water Resources), and information in Thurber Consultants library from previous involvement in the Murphy Creek Reservoir Area. A complete list of information pertinent to this project and reviewed for this work is included in the reference list (following Section 6 of the text).

The survey sectioning was carried out at 18 locations considered to be representative of some of the critical slopes around the reservoir. The locations of the sections are indicated on Fig. A-3.

TABLE A-1

SUMMARY OF SLOPE SECTION MAPPING
ON LEFT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR | | | EXISTING BACKSLOPE | | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) | | |
| 1 | 0.15 | 420 | 4:1 | 150 | 460 | 1:1 | 25 | - oversteep due to Keenlyside Dam discharge |
| 2 | 0.55 | 425 | 2:1 | 100 | 428 | 20:1 | 75 | |
| 3 | 1.25 | 423 | 4:1 | 75 | 427<br>431 | 20:1<br>1.5:1 | 50<br>150 | - armour from creek fan<br>- highway embankment |
| 4 | 2.45 | 425 | 15:1 | 2 | 428 | 1.5:1 | 75 | - highway embankment |
| 5 | 2.65 | 425 | 10:1 | 50 | 428 | 1.5:1 | 75 | - armour from creek fan<br>- highway embankment |
| 6 | 4.15 | 424 | 4:1 | 75 | 433 | 1.5:1 | 75 | - highway embankment |
| 7 | 5.40 | 424 | 3:1 | 100 | 428 | 1.5:1 | 75 | - highway embankment |
| 8 | 6.95 | 424 | 1.5:1 | 150 | 429 | 2:1 | 75 | - highway embankment |
| 9 | 7.20 | 422 | 10:1 | 125 | 427 | 1.5:1 | 75 | - adjacent to ferry landing<br>- fill backslope |
| 10 | 7.40 | - | - | - | 424<br>430 | 30:1<br>1:1 | 1<br>75 | - no apparent armour<br>- Pass Creek delta<br>- highway embankment |
| 11 | 7.90 | 422 | 2:1 | 75 | 424<br>426 | 1:1<br>100:1 | 50<br>5 | - Pass Creek fan |
| 12 | 8.15 | 422 | 3:1 | 100 | 426<br>429 | 1.5:1<br>1.5:1 | 100<br>100 | - 10 m wide roadway at 426 m |
| 13 | 8.20 | 422 | 3:1 | 100 | 429 | 2:1 | 100 | - edge of sewage lagoon |
| 14 | 8.45 | 421 | 8:1 | 1 | 429 | 4:1 | 5 | - sawmill debris present<br>- seepage at approx 425 m |
| 15 | 8.75 | 421 | 8:1 | 5 | 433 | 2:1 | 5 | - railway embankment |
| 16 | 8.95 | 425 | 2.5:1 | 75 | 435 | 1.5:1 | 10 | - residential property at crest |
| 17 | 9.10 | 425 | 2.5:1 | 75 | 437 | 1.5:1 | 75 | - fill from residential construction at crest |
| 18 | 9.25 | 425 | 2.5:1 | 50 | 437 | 1.5:1 | 50 | - houses within 5 m of crest |
| 19 | 9.35 | 420 | 5:1 | 1 | 440 | 1:1 | 2 | - tower for overhead gas pipeline crossing at crest |
| 20 | 9.50 | 420 | 5:1 | 1 | 440 | 1:1 | 2 | - oversteep, unstable slope |

* Refer to Fig. A-3, sheets 1 to 4 for location of sections

TABLE A-1 (continued)

SUMMARY OF SLOPE SECTION MAPPING
ON LEFT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR | | | EXISTING BACKSLOPE | | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) | | |
| 21 | 9.75 | 421 | 10:1 | 150 | 437 | 1.25:1 | 100 | - oversteep slope, cemetery at crest |
| Kootenay River - Right Bank | | | | | | | | |
| 22 | 0.60 | - | - | - | 424<br>437 | 1.25:1<br>2:1 | 25<br>75 | - armour not noted<br>- 300 m wide bench at 424 m |
| 23 | 1.00 | 423 | 2:1 | 175 | 426 | 1.25:1 | 25 | |
| 24 | 1.15 | 420 | Flat | Bedrock | 426 | 1:1 | 300 | - backslope rip rapped |
| 25 | 1.45 | 422 | 2:1 | 100 | 429 | 2:1 | 75 | |
| Kootenay River - Left Bank | | | | | | | | |
| 26 | 0.80 | 423 | - | 125 | 427 | 3:1 | 75 | - 400 m wide floodplain |
| 27 | 0.35 | - | - | - | 440 | 8:1 | 5 | - armour not apparent, old meander at 417 m |
| 28 | 0.25 | 423 | 4:1 | 300 | 440 | 8:1 | 100 | - massive armour to 2 m dia |
| 29 | 0.05 | 425 | 4:1 | 300 | 435 | 1.5:1 | 100 | - massive armour to 2.5 m dia |
| 30 | 10.60 | 424 | 4:1 | 350 | 435 | 1.5:1 | 125 | |
| 31 | 10.80 | 422 | 3:1 | 150 | 440 | 1.5:1 | 100 | - outfall from college at approx 419 m |
| 32 | 11.35 | 420 | 5:1 | 150 | 455 | 1.4:1 | 75 | - oversteep slope |
| 33 | 12.80 | 420 | 6:1 | 350 | 480 | 2:1 | 100 | |
| 34 | 12.95 | 423 | 5:1 | 300 | 480 | 2:1 | 125 | - seepage from base of slope |
| 35 | 13.10 | 423 | 4:1 | 300 | 475 | 1.5:1 | 100 | - massive armour to 4 m dia |
| 36 | 13.35 | 424 | 2.5:1 | 300 | 475 | 2:1 | 100 | |
| 37 | 13.60 | 422 | 6:1 | 200 | 445 | 5:1 | 5 | |
| 38 | 13.70 | 421 | 2:1 | 200 | 446 | 1.25:1 | 50 | - oversteep slope |
| 39 | 13.80 | 422 | 2:1 | 200 | 446 | 1.25:1 | 25 | - oversteep slope |

* Refer to Fig. A-3, sheets 4 and 5 for location of sections

TABLE A-1 (continued)

SUMMARY OF SLOPE SECTION MAPPING
ON LEFT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR | | | EXISTING BACKSLOPE | | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) | | |
| 40 | 14.70 | 421 | 5:1 | 100 | 440 | 1.5:1 | 50 | - cemetery at crest 0.4 to 0.6 km downstream |
| 41 | 15.80 | 421 | 3:1 | 100 | 435 | 1.5:1 | 50 | - houses near crest of slope |
| 42 | 16.40 | 420 | 5:1 | 50 | 428 | 4:1 | 50 | - towers for overhead powerline crossing near crest of slope, approx 0.1 km upstream |
| 43 | 18.15 | 421 | 5:1 | 50 | 428 | 1.5:1 | 50 | - towers for overhead powerline crossing near crest of slope approx 0.1 km up and downstream |
| 44 | 21.70 | 420 | 2.5:1 | | 428 | 1.5:1 | 75 | - cemetery at crest of slope |
| 45 | 23.10 | - | - | - | 456 | 1.25:1 | 15 | - no apparent armour<br>- oversteep slope |
| 46 | 23.65 | 418 | 3:1 | 75 | 450 | 1.25:1 | 25 | - oversteep slope |
| 47 | 27.65 | 418 | 2:1 | 75 | 460 | 1:25:1 | 50 | - oversteep slope |
| 48 | 29.45 | 413 | 2:1 | 75 | 429 | 1:25:1 | 75 | - oversteep slope |
| 49 | 30.95 | 415 | 3:1 | 100 | 465 | 1.4:1 | 25 | - oversteep slope |

* Refer to Fig. A-3, sheets 5 to 10 for location of section

## TABLE A-2

## SUMMARY OF SLOPE SECTION MAPPING ON RIGHT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR | | | EXISTING BACKSLOPE | | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) | | |
| 50 | 0.30 | 422 | 4:1 | 100 | 432 | 2:1 | 50 | - fill slope |
| 51 | 4.70 | 422 | 6:1 | 2 | 432 | 3:1 | 2 | - likely underlain by heavier armour |
| 52 | 5.35 | 422 | 6:1 | 2 | 426 | 3:1 | 2 | - likely underlain by heavier armour<br>- abandoned railroad grade at 426 m |
| | | | | | 430 | 3:1 | 2 | |
| 53 | 5.50 | 422 | 6:1 | 75 | 425 | 3:1 | 25 | - abandoned railroad grade at 425 m |
| | | | | | 430 | 3:1 | 2 | |
| 54 | 5.80 | 422 | 8:1 | 2 | 426 | 2:1 | 2 | - likely underlain by heavier armour<br>- steep slope due to creek erosion and/or past flooding |
| | | | | | 430 | 6:1 | 2 | |
| 55 | 6.40 | 422 | 7:1 | 75 | 424 | 2:1 | 2 | |
| | | | | | 433 | 1:1 | 2 | - steep slope resulting from past flooding |
| 56 | 7.05 | 424 | 3:1 | 75 | 430 | 1:1 | 2 | - oversteep slope |
| 57 | 7.65 | 424 | 2:1 | 100 | 430 | 3:1 | 75 | - buildings near crest |
| 58 | 8.10 | 423 | 2:1 | 150 | 430 | 1.5:1 | | - CP Rail bridge, directly under bridge, 300 mm dia rip rap to 424 m |
| 59 | 8.15 | 422 | 2:1 | 150 | 445 | 1.5:1 | 100 | - buildings near crest |
| 60 | 8.30 | 422 | 2:1 | 150 | 445 | 1.5:1 | 100 | - buildings near crest |
| 61 | 8.45 | 422 | 2:1 | 150 | 427 | 1:1 | 75 | - oversteep slope |
| | | | | | 430 | 4:1 | 75 | |
| 62 | 8.50 | 422 | 2:1 | 125 | 430 | 1.25:1 | 75 | - buildings at crest, slope is possibly fill |
| 63 | 8.60 | 423 | 2:1 | 125 | 430 | 1.5:1 | 75 | |
| 64 | 8.75 | 422 | 1.5:1 | 125 | 427 | 1.5:1 | 75 | |
| 65 | 8.90 | 425 | 1.5:1 | 200 | 427 | 1.5:1 | 75 | - placed rip rap<br>- houses at crest |
| 66 | 9.25 | 420 | 5:1 | 75 | 424 | 2:1 | 25 | - overhead gas pipeline crossing<br>- level at 424 m for 200 m |

* Refer to Fig. A-3, sheets 1 to 3 for location of sections

Table A-2 (continued)

SUMMARY OF SLOPE SECTION MAPPING
ON RIGHT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR | | | EXISTING BACKSLOPE | | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) | | |
| 67 | 10.35 | - | - | - | - | - | - | - buried gasline |
| 68 | 11.00 | 420 | 3:1 | 75 | | | | - overhead telephone cable crossing |
| | | | | | 435 | 2:1 | 75 | |
| 69 | 11.30 | 424 | 2.5:1 | 75 | | | | - armouring includes some massive boulders to 1.5 m |
| | | | | | 442 | 1.4:1 | 50 | - rubble fill,oversteep slope near crest |
| 70 | 11.55 | 421 | 4:1 | 75 | | | | |
| | | | | | 445 | 1.4:1 | 75 | - oversteep slope |
| 71 | 11.70 | 420 | 3:1 | 100 | | | | |
| | | | | | 423 | 1.5:1 | 75 | - 6 to 10 m wide bench at el 423 m |
| | | | | | 445 | 1.5:1 | 75 | |
| 72 | 12.00 | 423 | 5:1 | 300 | | | | |
| | | | | | 450 | 1.25:1 | 75 | - oversteep slope, buildings at crest |
| 73 | 12.15 | 424 | 2:1 | 150 | | | | |
| | | | | | 450 | 1.25:1 | 100 | - oversteep slope, buildings at crest |
| 74 | 12.25 | 424 | 3.5:1 | 175 | | | | |
| | | | | | 450 | 1.5:1 | 50 | |
| 75 | 12.55 | 426 | 2.5:1 | 125 | | | | |
| | | | | | 450 | 1.4:1 | 25 | - CMP leading into river at this location |
| 76 | 12.70 | 421 | 2.8:1 | 175 | | | | |
| | | | | | 470 | 1.4:1 | 75 | - oversteep slope |
| 77 | 12.90 | 421 | 2.8:1 | 175 | | | | - bench at elev 421 m |
| | | 422 | 1.5:1 | 75 | | | | |
| | | | | | 478 | 1.35:1 | 50 | - oversteep slope, buildings near crest |
| 78 | 13.05 | 421 | 2.4:1 | 150 | | | | - bench at elev 421 m |
| | | | | | 475 | 1.3:1 | 50 | - oversteep slope, buildings near crest |
| 79 | 13.20 | 421 | 1.5:1 | 100 | | | | |
| | | | | | 475 | 1.4:1 | 50 | - oversteep slope |
| 80 | 13.40 | 422 | 2:1 | 175 | | | | |
| | | 423 | 1.5:1 | 75 | | | | |
| | | | | | 472 | 1.4:1 | 75 | - oversteep slope |
| 81 | 13.80 | 421 | 3:1 | 200 | | | | - 4-5 m wide bench at 421 m |
| | | | | | 475 | 2:1 | 100 | |
| 82 | 14.15 | 421 | 3:1 | 100 | | | | |
| | | | | | 475 | 1.5:1 | 75 | - recent flume failure causing large erosional gulley |
| 83 | 14.30 | 421 | 2.8:1 | 150 | | | | |
| | | | | | 475 | 1.4:1 | 75 | - oversteep slope |
| 84 | 14.40 | 420 | 2:1 | 150 | | | | |
| | | 424 | 2:1 | 100 | | | | |
| | | | | | 475 | 1.4:1 | 50 | - oversteep slope |
| 85 | 14.45 | 418 | 2.3:1 | 150 | | | | |
| | | | | | 475 | 1.4:1 | 75 | - oversteep slope |

* Refer to Fig. A-3, sheets 3 and 5 for location of sections

Table A-2 (continued)

SUMMARY OF SLOPE SECTION MAPPING
ON RIGHT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR |  |  | EXISTING BACKSLOPE |  | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
|  |  | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) |  |  |
| 86 | 14.65 | 419 | 2.7:1 | 125 | 475 | 1.4:1 | 75 | - oversteep slope |
| 87 | 14.85 | 420 | 2.5:1 | 150 | 473 | 1.3:1 | 100 | - oversteep slope, buildings at crest |
| 88 | 15.00 | 420 | 2.7:1 | 150 | 428 | 2:1 | 100 | - roadway at crest |
| 89 | 15.55 | 420 | 8:1 | 1 | 425 | 2:1 | 5 | - seepage at crest<br>- roadway at crest |
| 90 | 15.85 | 420 | 2.7:1 | 150 | 424 | 1.25:1 | 75 | - oversteep slope, roadway at crest |
| 91 | 15.95 | 421 | 2.7:1 | 150 | 465 | 1.25:1 | 50 | - oversteep slope, seepage near base |
| 92 | 16.45 | 421 | 2:1 | 75 | 424<br>455 | 1.4:1<br>2:1 | 50<br>50 | - bench at 424 m<br>- buildings at crest |
| 93 | 16.85 | 424 | 4:1 | 75 | 450 | 1.5:1 | 100 | - creek fan |
| 94 | 16.95 | 422 | 2.7:1 | 125 | 428<br>455 | 2:1<br>1.5:1 | 100<br>100 | - bench at el 428 m |
| 95 | 17.05 | 422 | 2.7:1 | 175 | 433 | 1.5:1 | 125 |  |
| 96 | 17.15 | 422 | 3:1 | 175 | 433 | 2:1 | 100 |  |
| 97 | 17.25 | 419 | 5:1 | 175 | 433 | 2:1 | 75 |  |
| 98 | 17.95 | - | - | - | 485 | 1.5:1 | 5 | - no apparent armour<br>- railgrade at el 485 m |
| 99 | 18.25 | 418 | 8:1 | 75 | 428<br>475 | 2:1<br>1.7:1 | 5<br>5 | - towers for overhead powerline crossing at crest |
| 100 | 18.45 | 420 | 4:1 | 150 | 465 | 1.4:1 | 100 | - oversteep slope |
| 101 | 18.55 | 418 | 3:1 | 200 | 465 | 1.25:1 | 150 | - severe current<br>- oversteep slope |
| 102 | 18.65 | 422 | 3:1 | 300 | 468 | 1.25:1 | 100 | - oversteep slope |
| 103 | 19.70 | 418 | 3:1 | 200 | 470 | 1.35:1 | 75 | - oversteep slope |
| 104 | 20.05 | 421 | 10:1 | 300 | - | - | - | - Blueberry Creek delta |
| 105 | 20.25 | 423 | 4:1 | 75 | 440 | 2:1 | 75 |  |

* Refer to Fig. A-3, sheets 5 to 7 for location of sections

Table A-2 (continued)

SUMMARY OF SLOPE SECTION MAPPING
ON RIGHT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR | | | EXISTING BACKSLOPE | | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) | | |
| 106 | 20.45 | 418 | 4:1 | 200 | 465 | 1.4:1 | 75 | - oversteep slope |
| 107 | 20.65 | - | - | - | 470 | 1.4:1 | 75 | - no apparent armour<br>- oversteep slope |
| 108 | 20.70 | - | - | - | 470 | 1.4:1 | 75 | - no apparent armour<br>- oversteep slope |
| 109 | 20.75 | - | - | - | 470 | 1.4:1 | 75 | - no apparent armour<br>- oversteep slope |
| 110 | 20.85 | 418 | 4:1 | 150 | 470 | 1.25:1 | 50 | - oversteep slope |
| 111 | 21.15 | 419 | 3:1 | 125 | 424<br>470 | 1.5:1<br>1.3:1 | 50<br>50 | - narrow bench at elev 424 m<br>- oversteep slope |
| 112 | 21.25 | 417 | 4:1 | 75 | 470 | 1.25:1 | 50 | - oversteep slope |
| 113 | 21.40 | 417 | 4:1 | 75 | 470 | 1.25:1 | 50 | - oversteep slope |
| 114 | 21.80 | 423 | 3:1 | 200 | 465 | 1.4:1 | 75 | - oversteep slope |
| 115 | 21.85 | 424 | 3:1 | 150 | 467 | 1.4:1 | 100 | - oversteep slope |
| 116 | 21.95 | 415 | 3:1 | 100 | 470 | 1.3:1 | 50 | - oversteep slope |
| 117 | 22.00 | 420 | 3:1 | 100 | 470 | 1.3:1 | 100 | - oversteep slope |
| 118 | 22.05 | 420 | 5:1 | 100 | 475 | 1.3:1 | 75 | - oversteep slope |
| 119 | 22.20 | 418 | 4:1 | 75 | 475 | 1.4:1 | 75 | - oversteep slope<br>railroad near crest |
| 120 | 22.30 | 418 | 4:1 | 75 | 475 | 1.3:1 | 75 | - oversteep slope<br>railroad near crest |
| 121 | 22.35 | 417 | 4:1 | 75 | 475 | 1.3:1 | 100 | - oversteep slope<br>railroad near crest |
| 122 | 22.45 | 416 | 5:1 | 75 | 475 | 1.3:1 | 125 | - oversteep slope<br>railroad near crest |
| 123 | 22.55 | 416 | 5:1 | 300 | 427<br>475 | 1.6:1<br>1.3:1 | 75<br>75 | - bench at el 427 m<br>- railroad near crest |
| 124 | 22.70 | 415 | 5:1 | 100 | 473 | 1.3:1 | 10 | - oversteep slope |
| 125 | 22.75 | 418 | 8:1 | 150 | 472 | 1.2:1 | 10 | - armour from China Ck fan<br>- unstable slope |
| 126 | 23.05 | 417 | 8:1 | 100 | 445 | 20:1 | 100 | - China Creek fan |

* Refer to Fig. A-3, sheets 7 and 8 for location of sections

TABLE A-2 (continued)

## SUMMARY OF SLOPE SECTION MAPPING ON RIGHT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR | | | EXISTING BACKSLOPE | | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) | | |
| 127 | 23.40 | 415 | 10:1 | 50 | 435 | 10:1 | 50 | - rail grade at el 435 m |
| 128 | 23.60 | 417 | 8:1 | 100 | 425 | 20:1 | 100 | |
| | | | | | 434 | 1.5:1 | 100 | - rail grade at el 434 m |
| 129 | 23.70 | 417 | 10:1 | 100 | 425 | 20:1 | 75 | |
| | | | | | 434 | 3:1 | 50 | - rail grade at el 434 m |
| 130 | 23.75 | 417 | 10:1 | 100 | 425 | 20:1 | 75 | |
| | | | | | 434 | 5:1 | 75 | - rail grade at el 434 m |
| 131 | 23.95 | 417 | 10:1 | 100 | 425 | 20:1 | 75 | |
| | | | | | 434 | 4:1 | 75 | - rail grade at el 434 m |
| 132 | 24.10 | 417 | 10:1 | 100 | 420 | 20:1 | 75 | |
| | | | | | 435 | 4:1 | 100 | - rail grade at el 435 m |
| 133 | 25.20 | armour not noted | | | 425 | 1.5:1 | 100 | |
| | | | | | 430 | 20:1 | 100 | |
| 134 | 25.30 | armour not noted | | | 426 | 1.5:1 | 100 | |
| | | | | | 430 | 12:1 | 100 | |
| 135 | 25.40 | armour not noted | | | 430 | 2:1 | 75 | - level bench at el 430 m |
| 136 | 25.50 | armour not noted | | | 430 | 2:1 | 25 | - level bench at el 430 m |
| 137 | 26.00 | armour not noted | | | 430 | 2:1 | 75 | - level bench at el 430 m |
| 138 | 26.05 | armour not noted | | | 425 | 2:1 | 100 | |
| | | | | | 430 | 10:1 | 100 | |
| 139 | 26.05 | armour not noted | | | 425 | 2:1 | 100 | |
| | | | | | 430 | 8:1 | 125 | |
| 140 | 26.10 | armour not noted | | | 430 | 1.5:1 | Bedrock | - 100 m wide bench at crest |
| 141 | 26.15 | armour not noted | | | 425 | 1.5:1 | 25 | |
| | | | | | 429 | 10:1 | 25 | - rail grade 10 m from crest |
| 142 | 26.20 | armour not noted | | | 429 | 1.1:1 | 75 | - oversteep slope, rail grade on 13 m wide bench at crest |
| 143 | 26.30 | armour not noted | | | 429 | 1.4:1 | 125 | - rail grade on 20 m wide bench at crest |
| 144 | 26.35 | armour not noted | | | 429 | 1.4:1 | 125 | - rail grade on 24 m wide bench at crest |

* Refer to Fig. A-3, sheets 8 and 9 for location of sections

TABLE A-2 (continued)

SUMMARY OF SLOPE SECTION MAPPING
ON RIGHT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR |  |  | EXISTING BACKSLOPE |  | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
|  |  | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) |  |  |
| 145 | 26.45 |  | armour not noted |  | 429 | 1.5:1 | 175 | - rail grade on 8 m wide bench at crest |
| 146 | 26.50 | 413 | 2:1 | 125 | 429 | 1.2:1 | 125 | - oversteep slope, rail grade on 6 m wide bench at crest |
| 147 | 26.60 | 413 | 2.5:1 | 125 | 429 | 1.1:1 | 125 | - oversteep slope, rail grade on 8 m wide bench at crest |
| 148 | 26.70 | 413 | 1.5:1 | 150 | 429 | 1.1:1 | 150 | - oversteep slope, rail grade on 6 m wide bench at crest |
| 149 | 26.80 |  | armour not noted |  | 427<br>429 | 1:1<br>2:1 | 150<br>75 | - oversteep slope<br>- rail grade on 9 m wide bench at crest |
| 150 | 26.90 |  | armour not noted |  | 424<br>429 | 1.5:1<br>4:1 | 100<br>25 | - rail grade on 5 m wide bench at crest |
| 151 | 27.00 | - | - | - | 429 | 1.4:1 | 25 | - no apparent armour<br>- 20-24 m wide bench at crest |
| 152 | 27.10 | - |  | - | 429 | 1.4:1 | 25 | - no apparent armour<br>- 20-24 m wide bench at crest |
| 153 | 27.20 | 420 | 4:1 | 200 | 428 | 1.7:1 | 25 | - 18-20 m wide bench at crest |
| 154 | 27.30 | 420 | 4:1 | 200 | 427 | 2:1 | 50 | - 8-10 m wide bench at crest |
| 155 | 27.40 | - | - | - | 427 | 2:1 | 125 | - no apparent armour<br>- 7-10 m wide bench at crest |
| 156 | 27.50 | 424 |  | Bedrock | 428 | 2:1 | 125 |  |
| 157 | 27.60 | - | - | Bedrock | - | - | Bedrock | - Rail grade in rock cut at el 427 m |
| 158 | 27.65 | 419 | 4:1 | 100 | 427 | 1.7:1 | 125 | - 9-12 m wide bench at crest |
| 159 | 27.75 | 419 | 4:1 | 100 | 426 | 1.2:1 | 75 | - 6-8 m wide bench at crest |
| 160 | 27.85 | 419 | 4:1 | 150 | 426 | 1.7:1 | 125 | - 9-12 m wide bench at crest |
| 161 | 27.95 | 419 | 4:1 | 200 | 426 | 1.6:1 | 150 | - 7-10 m wide bench at crest |
| 162 | 28.05 | 419 | 4:1 | 250 | 425 | 1.4:1 | 150 | - 5-8 m wide bench at crest |

* Refer to Fig. A-3, sheet 9 for location of sections

TABLE A-2 (continued)

SUMMARY OF SLOPE SECTION MAPPING
ON RIGHT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR | | | EXISTING BACKSLOPE | | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) | | |
| 163 | 28.20 | 419 | 4:1 | 200 | 425 | 1.4:1 | 150 | - 14-16 m wide bench at crest |
| 164 | 28.30 | 419 | 4:1 | 100 | 425 | 1.6:1 | 125 | - 18-22 m wide bench at crest |
| 165 | 28.40 | 419 | 3:1 | 100 | 424 | 1.7:1 | 150 | - 14-18 m wide bench at crest |
| 166 | 28.50 | 419 | 2:1 | 100 | 424 | 2:1 | 100 | - 12-14 m wide bench at crest |
| 167 | 28.60 | armour not noted | | | 430* | 1.5:1 | 100 | |
| 168 | 28.70 | armour not noted | | | 435* | 1.5:1 | 100 | |
| 169 | 28.80 | armour not noted | | | 450* | 1.4:1 | 100 | - oversteep slope |
| 170 | 28.90 | armour not noted | | | 460* | 1.4:1 | 100 | - oversteep slope |
| 171 | 29.00 | armour not noted | | | 460* | 1.4:1 | 100 | - oversteep slope |
| 172 | 29.10 | armour not noted | | | 460* | 1.4:1 | 100 | - oversteep slope |
| 173 | 29.20 | armour not noted | | | 460* | 1.4:1 | 100 | - oversteep slope |
| 174 | 29.30 | armour not noted | | | 460* | 1.4:1 | 100 | - oversteep slope |
| 175 | 29.35 | armour not noted | | | 460* | 1.4:1 | 100 | - oversteep slope |
| 176 | 29.45 | armour not noted | | | 460* | 1.4:1 | 100 | - oversteep slope |
| 177 | 29.50 | armour not noted | | | 460* | 1.4:1 | 100 | - oversteep slope |
| 178 | 29.80 | armour not noted | | | 455* | - | Bedrock | - outcrop extends 40 m south, 10 m north |
| 179 | 29.90 | armour not noted | | | 455 | - | 125 | |
| 180 | 30.00 | armour not noted | | | 419<br>422<br>425<br>455 | 1.4:1<br>1.7:1<br>6:1<br>1.4:1 | Bedrock<br>75<br>125<br>125 | - bedrock extends 20 m north<br>- 8-10 m wide bench at crest |
| 181 | 30.20 | armour not noted | | | 423<br>455 | -<br>1.3:1 | Bedrock<br>125 | - 6-8 m wide bench at crest<br>- bedrock extends 5 m south and 25 m north |

* Rail Grade inundated, existing backslope refers to upper slope with toe at rail grade elevation.

** Refer to Fig. A-3, sheets 9 and 10 for location of sections

## TABLE A-2 (continued)

### SUMMARY OF SLOPE SECTION MAPPING ON RIGHT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR |  |  | EXISTING BACKSLOPE |  | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
|  |  | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) |  |  |
| 182 | 30.20 |  | armour not noted |  | 423<br>460 | 1.4:1<br>1.4:1 | 50<br>50 | - 9-10 m wide bench at crest<br>- bedrock noted 30 m north at el 427 |
| 183 | 30.25 |  | armour not noted |  | 423<br>463 | 1.4:1<br>1.4:1 | 50<br>50 | - 8-10 m wide bench at crest<br>- limit to oversteep slope |
| 184 | 30.30 |  | armour not noted |  | 422<br>463 | 1.4:1<br>1.4:1 | 50<br>50 | - 8-10 m wide bench at crest<br>- limit to oversteep slope |
| 185 | 30.40 | 416 | 3:1 | 200 | 422<br>460 | 10:1<br>1.7:1 | 125<br>75 | - 8 m wide bench at crest |
| 186 | 30.50 | 416 | 3:1 | 200 | 423<br>458 | 10:1<br>1.7:1 | 125<br>150 | - 10 m wide bench at crest |
| 187 | 30.60 | 416 | 5:1 | 250 | 423<br>455 | 10:1<br>2:1 | 125<br>125 | - 11-20 m wide bench at crest |
| 188 | 30.75 |  | armour not noted |  | 422<br>455 | 10:1<br>1.8:1 | 125<br>125 | - rail grade at crest |
| 189 | 30.85 | - | - | - | 422 | 8:1 | 100 | - no apparent armour<br>- rail grade at crest |
| 190 | 30.95 | - | - | - | 422<br>465 | 5:1<br>1.4:1 | 100<br>50 | - no apparent armour<br>- rail grade at crest<br>- oversteep slope |
| 191 | 31.05 | - | - | - | 422<br>465 | 3:1<br>1.5:1 | 125<br>50 | - no apparent armour<br>- rail grade at crest |
| 192 | 31.15 |  | armour not noted |  | 422<br>470 | 2.5:1<br>1.5:1 | 125<br>150 | - rail grade at crest |
| 193 | 31.25 |  | armour not noted |  | 422<br>470 | 1.5:1<br>1.5:1 | 100<br>100 | - rail grade at crest |
| 194 | 31.35 | 415 | 2:1 | 200 | 422<br>465 | 1.5:1<br>1.5:1 | 125<br>125 | - rail grade at crest |
| 195 | 31.45 |  | armour not noted |  | 422<br>465 | 1.7:1<br>1.3:1 | 25<br>50 | - rail grade at crest<br>- oversteep slope |
| 196 | 31.55 | 419 | 1.5:1 | 100 | 423<br>465 | 1.7:1<br>1.3:1 | 50<br>125 | - rail grade at crest<br>- oversteep slope |
| 197 | 31.65 | 415 | 3:1 | 150 | 423<br>460 | 2:1<br>1.3:1 | 50<br>50 | - rail grade at crest<br>- oversteep slope |
| 198 | 31.75 | 421 | 4:1 | 200 | 422<br>425 | 20:1<br>10:1 | 75<br>75 | - rail grade at crest |

* Refer to Fig. A-3, sheet 10 for location of sections

TABLE A-2 (continued)

SUMMARY OF SLOPE SECTION MAPPING
ON RIGHT BANK

| SECTION NUMBER* | SECTION LOCATION (km) | EXISTING RIVER ARMOUR | | | EXISTING BACKSLOPE | | ESTIMATED LONG TERM BEACH SLOPE MATERIAL $D_{50}$ (mm) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | MAXIMUM ELEV. (m) | SLOPE horiz:vert | ESTIMATED $D_{50}$ (mm) | CREST ELEV. (m) | SLOPE (horiz:vert) | | |
| 199 | 31.85 | 420 | 4:1 | 200 | 425 | 10:1 | 50 | - 2 m high railroad fill embankment at el 420 m on slope |
| 200 | 31.95 | 418 | 5:1 | 200 | 422 | 1.5:1 | 50 | - railroad fill, grade at crest |
| | | | | | 435 | 1.3:1 | 125 | - oversteep slope |
| 201 | 32.05 | 415 | 2:1 | 200 | 421 | 2:1 | 100 | - railroad fill, grade at crest |
| | | | | | 435 | 1.3:1 | 100 | - oversteep slope |
| 202 | 32.15 | 415 | 2:1 | 200 | 421 | 2:1 | 100 | - railroad fill, grade at crest |
| | | | | | 430 | 1.4:1 | 150 | - oversteep slope |
| 203 | 32.25 | 415 | 2:1 | 300 | 421 | 2:1 | 100 | - railroad fill, grade at crest |
| | | | | | 430 | 1.4:1 | 75 | - oversteep slope |
| 204 | 32.35 | 416 | 2:1 | 300 | 421 | 2:1 | 150 | - railroad fill, grade at crest |
| | | | | | 435 | 1.5:1 | 100 | |
| 205 | 32.45 | 417 | 2.7:1 | 200 | 421 | 2:1 | 75 | - railroad fill, grade at crest |
| | | | | | 437 | 1.5:1 | 75 | |
| 206 | 32.50 | 422 | - | Bedrock | 422 | 2:1 | 150 | - rail grade at crest |
| | | | | | 432 | 2:1 | 150 | |
| 207 | 32.60 | - | - | Bedrock | 430 | 4:1 | 75 | - rail grade at 422 m |
| 208 | 32.70 | - | - | Bedrock | - | - | Bedrock | - rail grade at 423 m |
| 209 | 32.80 | - | - | Bedrock | - | - | Bedrock | - rail grade at 423 m |
| 210 | 32.90 | - | - | Bedrock | - | - | Bedrock | - rail grade at 424 m |
| 211 | 33.00 | - | - | Bedrock | - | - | Bedrock | - rail grade at 425 m |
| 212 | 33.10 | - | - | Bedrock | - | - | Bedrock | - rail grade at 426 m |
| 213 | 33.20 | - | - | Bedrock | - | - | Bedrock | - rail grade at 427 m |
| 214 | 33.30 | - | - | Bedrock | - | - | Bedrock | - rail grade at 428 m |
| 215 | 33.40 | - | - | Bedrock | - | - | Bedrock | - rail grade at 429 m |
| 216 | 33.50 | - | - | Bedrock | - | - | Bedrock | - rail grade at 431 m |

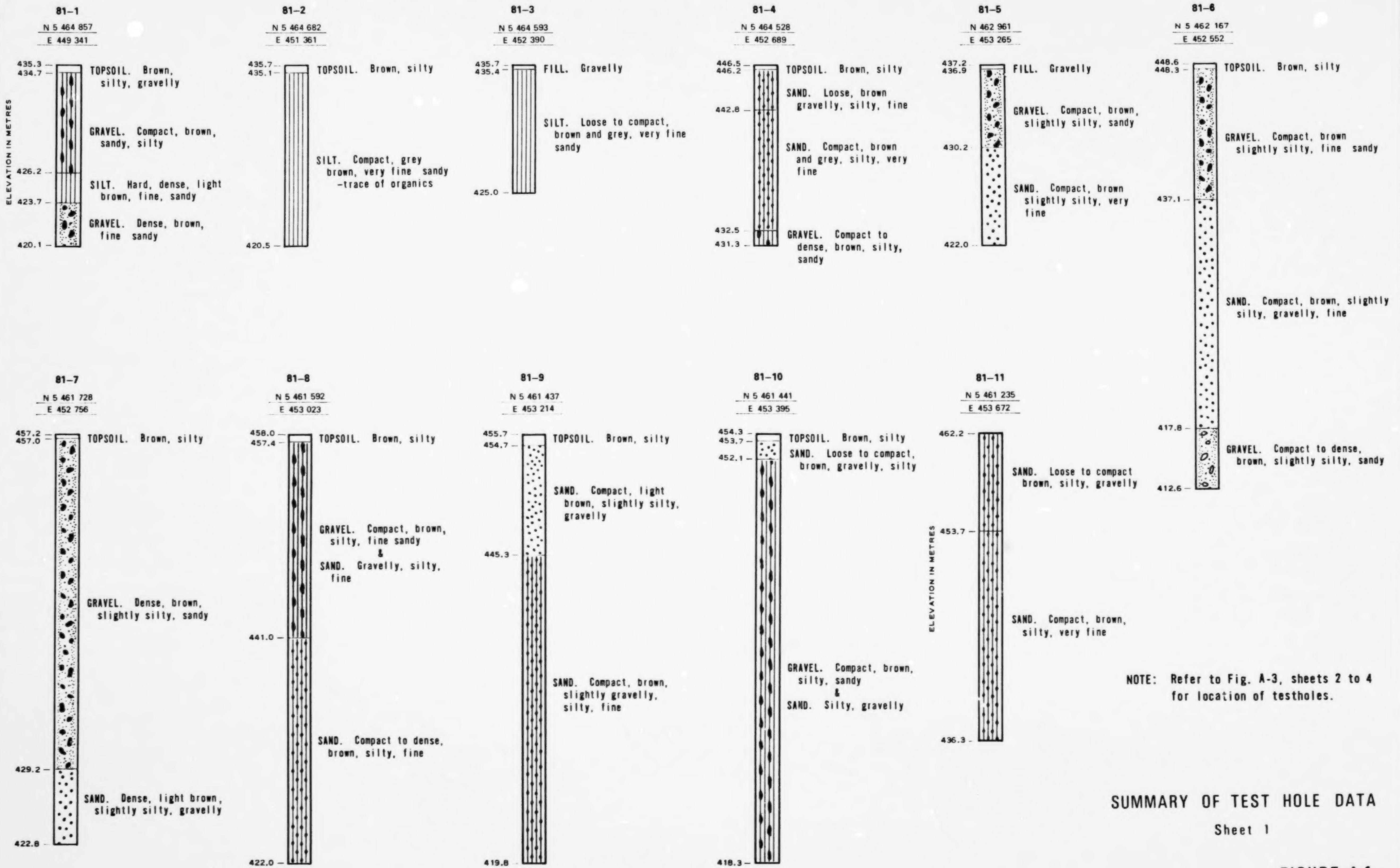
* Refer to Fig. A-3, sheet 10 for location of section

## TABLE A-3

### SUMMARY OF PIEZOMETER MEASUREMENTS

| TEST HOLE NUMBER | COLLAR ELEV. (m) | PIEZOMETER TIP ELEV. (m) | PIEZOMETRIC ELEV. (m) | | | | |
|---|---|---|---|---|---|---|---|
| | | | JULY 3/81 | JULY 10/81 | OCT. 17/81 | NOV. 4/81 | DEC. 9 TO 13/81 |
| 81-1 | 435.2 | 420.4 | 420.9 | 421.5 | dry | | dry |
| 81-2 | 435.6 | 420.3 | 423.1 | 423.4 | 422.5 | | 423.0 |
| 81-3 | 435.7 | 425.1 | dry | dry | dry | | dry |
| 81-4 | 446.5 | 431.2 | dry | dry | dry | | dry |
| 81-5 | 437.1 | 421.9 | dry | dry | dry | | dry |
| 81-6 | 448.5 | 412.5 | 419.4 | 420.1 | 416.6 | | 418.1 |
| 81-7 | 457.2 | 420.8 | dry | dry | dry | | dry |
| 81-8 | 458.0 | 422.0 | dry | dry | | | dry |
| 81-9 | 455.6 | 419.7 | dry | 420.3 | | | dry |
| 81-10 | 454.3 | 419.5 | dry | 420.4 | dry | | dry |
| 81-11 | 462.2 | 436.3 | dry | dry | dry | | dry |
| 81-12 | 483.3 | 426.9 | dry | dry | dry | | dry |
| 81-13 | 475.9 | 424.4 | dry | dry | dry | | dry |
| 81-14 | 474.5 | 442.8 | dry | dry | dry | | dry |
| 81-15 | 445.7 | 425.6 | dry | dry | | | |
| 81-16 | 443.8 | 417.6 | 419.6 | 420.3 | dry | | 418.2 |
| 81-17 | 450.8 | 404.7 | | 418.5 | 420.4 | | 417.6 |
| 81-18* | | | | | | | |
| 81-19 | 469.2 | 418.3 | dry | 419.3 | 419.6 | | 419.4 |
| 81-20 | 476.5 | 421.3 | dry | dry | | | dry |
| 81-21 | 466.1 | 419.8 | | dry | dry | | dry |
| 81-22 | 469.8 | 424.0 | | dry | | | dry |
| 81-23 | 456.4 | 422.3 | | dry | dry | | dry |
| 81-24 | 516.4 | 454.4 | dry | dry | | | dry |
| 81-25 | 506.9 | 455.7 | | dry | dry | | dry |
| 81-26 | 465.5 | 430.4 | | | | | dry |
| 81-27 | 469.6 | 434.5 | | dry | dry | | dry |
| 81-28 | 466.9 | 426.1 | | dry | dry | | dry |
| 81-29 | 436.3 | 419.9 | 420.6 | 421.1 | 420.2 | 420.3 | dry |
| 81-30 | 428.0 | 413.9 | 420.3 | 421.3 | 418.5 | 418.2 | 418.9 |
| 81-31 | 427.7 | 415.8 | 419.9 | 420.0 | 419.4 | 419.1 | 419.2 |
| 81-32 | 427.7 | 418.1 | 420.1 | 420.2 | 419.8 | 419.8 | 419.6 |
| 81-33 | 429.8 | 417.7 | 419.9 | 420.4 | 419.2 | 418.9 | 418.9 |
| 81-34 | 432.0 | 418.0 | 419.5 | 419.8 | 419.1 | 418.9 | 418.8 |
| 81-35 | 430.5 | 418.9 | 419.9 | 420.1 | dry | dry | 419.2 |
| 81-36 | 432.1 | 418.1 | 419.9 | 420.2 | | | 418.9 |
| 81-37 | 435.1 | 416.2 | 419.9 | 420.2 | 418.9 | 418.9 | 418.9 |
| 81-38 | 427.1 | 418.0 | 419.9 | 420.4 | dry | dry | 418.7 |
| 81-39 | 435.9 | 417.2 | 419.5 | | 418.2 | 417.9 | 418.2 |
| 81-40 | 436.9 | 419.7 | dry | dry | 420.2 | dry | 420.1 |

* TH 81-18 deleted from drilling program.

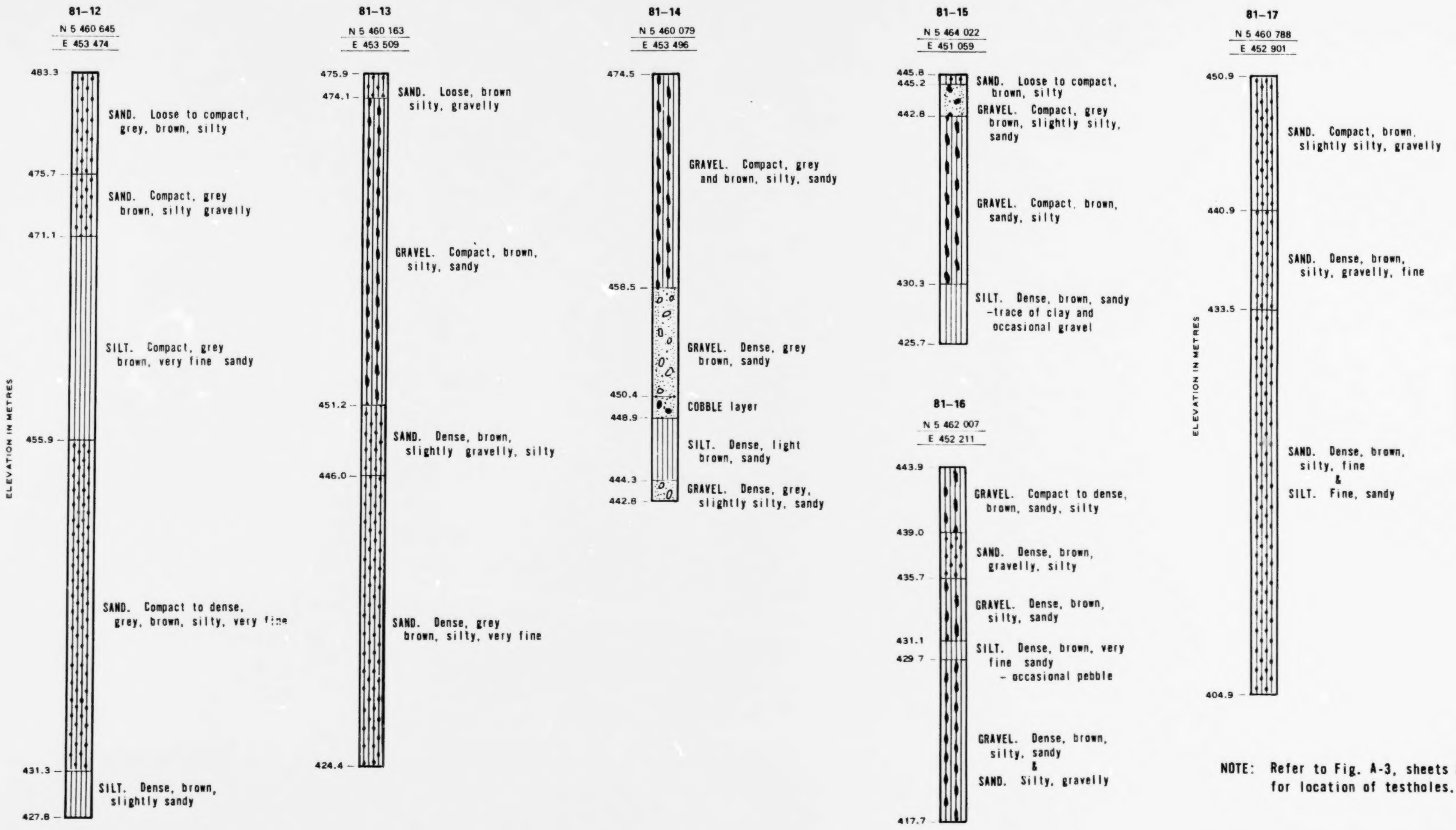

81–1
N 5 464 857
E 449 341
435.3
434.7
TOPSOIL. Brown, silty, gravelly
ELEVATION IN METRES
GRAVEL. Compact, brown, sandy, silty
426.2
SILT. Hard, dense, light brown, fine, sandy
423.7
GRAVEL. Dense, brown, fine sandy
420.1
81–2
N 5 464 682
E 451 361
435.7
435.1
TOPSOIL. Brown, silty
SILT. Compact, grey brown, very fine sandy -trace of organics
420.5
81–3
N 5 464 593
E 452 390
435.7
435.4
FILL. Gravelly
SILT. Loose to compact, brown and grey, very fine sandy
425.0
81–4
N 5 464 528
E 452 689
446.5
446.2
TOPSOIL. Brown, silty
SAND. Loose, brown gravelly, silty, fine
442.8
SAND. Compact, brown and grey, silty, very fine
432.5
431.3
GRAVEL. Compact to dense, brown, silty, sandy
81–5
N 462 961
E 453 265
437.2
436.9
FILL. Gravelly
GRAVEL. Compact, brown, slightly silty, sandy
430.2
SAND. Compact, brown slightly silty, very fine
422.0
81–6
N 5 462 167
E 452 552
448.6
448.3
TOPSOIL. Brown, silty
GRAVEL. Compact, brown slightly silty, fine sandy
437.1
SAND. Compact, brown, slightly silty, gravelly, fine
417.8
GRAVEL. Compact to dense, brown, slightly silty, sandy
412.6
81–7
N 5 461 728
E 452 756
457.2
457.0
TOPSOIL. Brown, silty
GRAVEL. Dense, brown, slightly silty, sandy
429.2
SAND. Dense, light brown, slightly silty, gravelly
422.8
81–8
N 5 461 592
E 453 023
458.0
457.4
TOPSOIL. Brown, silty
GRAVEL. Compact, brown, silty, fine sandy & SAND. Gravelly, silty, fine
441.0
SAND. Compact to dense, brown, silty, fine
422.0
81–9
N 5 461 437
E 453 214
455.7
454.7
TOPSOIL. Brown, silty
SAND. Compact, light brown, slightly silty, gravelly
445.3
SAND. Compact, brown, slightly gravelly, silty, fine
419.8
81–10
N 5 461 441
E 453 395
454.3
453.7
TOPSOIL. Brown, silty
SAND. Loose to compact, brown, gravelly, silty
452.1
GRAVEL. Compact, brown, silty, sandy & SAND. Silty, gravelly
418.3
81–11
N 5 461 235
E 453 672
462.2
SAND. Loose to compact brown, silty, gravelly
453.7
ELEVATION IN METRES
SAND. Compact, brown, silty, very fine
436.3

NOTE: Refer to Fig. A-3, sheets 2 to 4 for location of testholes.

SUMMARY OF TEST HOLE DATA

Sheet 1

FIGURE A-1

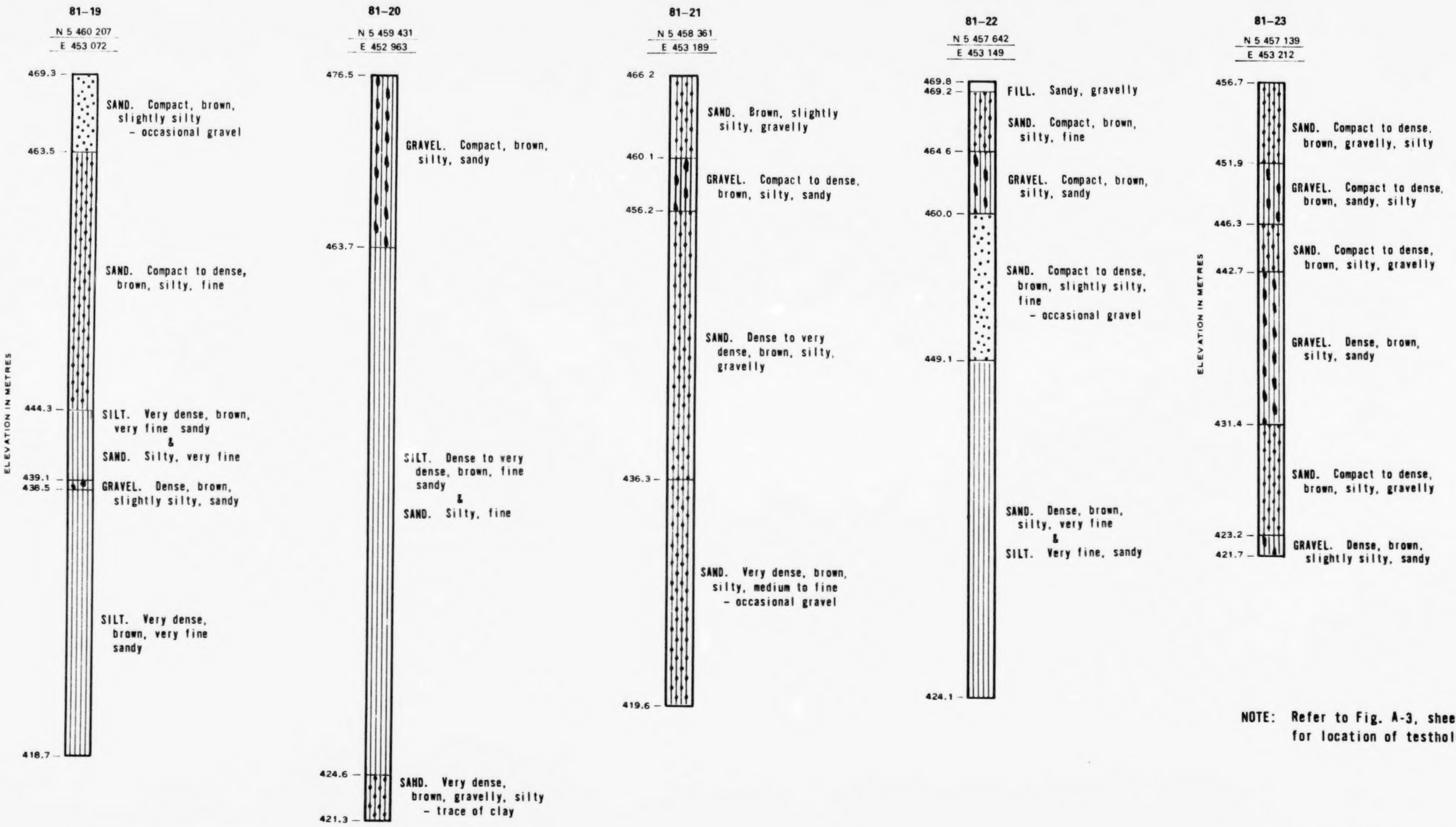

81-12
N 5 460 645
E 453 474
483.3
SAND. Loose to compact, grey, brown, silty
475.7
SAND. Compact, grey brown, silty gravelly
471.1
SILT. Compact, grey brown, very fine sandy
455.9
SAND. Compact to dense, grey, brown, silty, very fine
431.3
SILT. Dense, brown, slightly sandy
427.8
ELEVATION IN METRES
81-13
N 5 460 163
E 453 509
475.9
474.1
SAND. Loose, brown silty, gravelly
GRAVEL. Compact, brown, silty, sandy
451.2
SAND. Dense, brown, slightly gravelly, silty
446.0
SAND. Dense, grey brown, silty, very fine
424.4
81-14
N 5 460 079
E 453 496
474.5
GRAVEL. Compact, grey and brown, silty, sandy
458.5
GRAVEL. Dense, grey brown, sandy
450.4
COBBLE layer
448.9
SILT. Dense, light brown, sandy
444.3
GRAVEL. Dense, grey, slightly silty, sandy
442.8
81-15
N 5 464 022
E 451 059
445.8
445.2
SAND. Loose to compact, brown, silty
GRAVEL. Compact, grey brown, slightly silty, sandy
442.8
GRAVEL. Compact, brown, sandy, silty
430.3
SILT. Dense, brown, sandy -trace of clay and occasional gravel
425.7
81-16
N 5 462 007
E 452 211
443.9
GRAVEL. Compact to dense, brown, sandy, silty
439.0
SAND. Dense, brown, gravelly, silty
435.7
GRAVEL. Dense, brown, silty, sandy
431.1
SILT. Dense, brown, very fine sandy - occasional pebble
429 7
GRAVEL. Dense, brown, silty, sandy & SAND. Silty, gravelly
417.7
81-17
N 5 460 788
E 452 901
450.9
SAND. Compact, brown, slightly silty, gravelly
440.9
SAND. Dense, brown, silty, gravelly, fine
433.5
ELEVATION IN METRES
SAND. Dense, brown, silty, fine & SILT. Fine, sandy
404.9

NOTE: Refer to Fig. A-3, sheets 3 and 5 for location of testholes.

SUMMARY OF TEST HOLE DATA

Sheet 2

FIGURE A-1

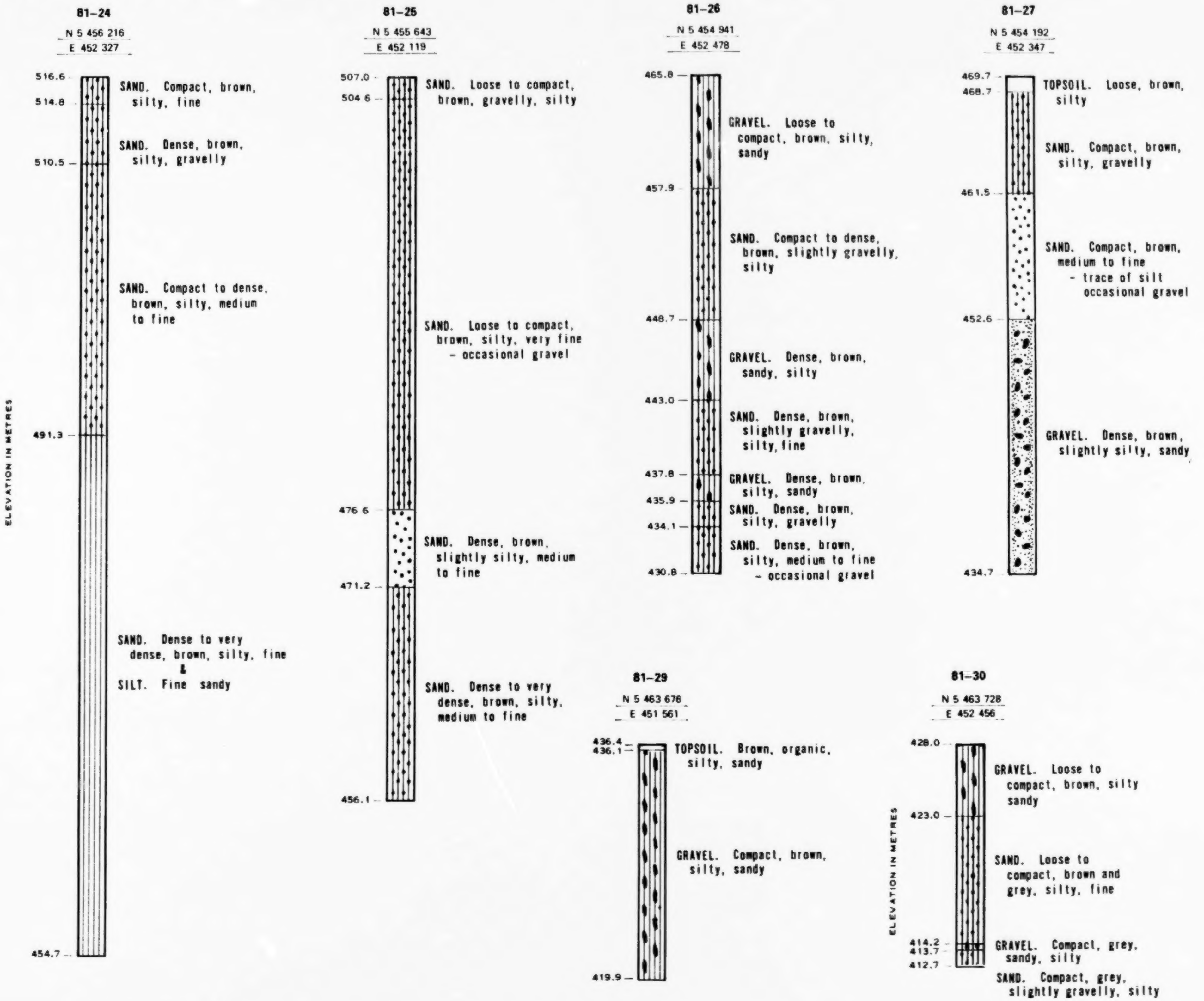

81–19
N 5 460 207
E 453 072
469.3
SAND. Compact, brown, slightly silty - occasional gravel
463.5
SAND. Compact to dense, brown, silty, fine
444.3
SILT. Very dense, brown, very fine sandy & SAND. Silty, very fine
439.1
438.5
GRAVEL. Dense, brown, slightly silty, sandy
SILT. Very dense, brown, very fine sandy
418.7
ELEVATION IN METRES
81–20
N 5 459 431
E 452 963
476.5
GRAVEL. Compact, brown, silty, sandy
463.7
SILT. Dense to very dense, brown, fine sandy & SAND. Silty, fine
424.6
SAND. Very dense, brown, gravelly, silty - trace of clay
421.3
81–21
N 5 458 361
E 453 189
466.2
SAND. Brown, slightly silty, gravelly
460.1
GRAVEL. Compact to dense, brown, silty, sandy
456.2
SAND. Dense to very dense, brown, silty, gravelly
436.3
SAND. Very dense, brown, silty, medium to fine - occasional gravel
419.6
81–22
N 5 457 642
E 453 149
469.8
469.2
FILL. Sandy, gravelly
SAND. Compact, brown, silty, fine
464.6
GRAVEL. Compact, brown, silty, sandy
460.0
SAND. Compact to dense, brown, slightly silty, fine - occasional gravel
449.1
SAND. Dense, brown, silty, very fine & SILT. Very fine, sandy
424.1
81–23
N 5 457 139
E 453 212
456.7
SAND. Compact to dense, brown, gravelly, silty
451.9
GRAVEL. Compact to dense, brown, sandy, silty
446.3
SAND. Compact to dense, brown, silty, gravelly
442.7
GRAVEL. Dense, brown, silty, sandy
431.4
SAND. Compact to dense, brown, silty, gravelly
423.2
421.7
GRAVEL. Dense, brown, slightly silty, sandy
ELEVATION IN METRES

NOTE: Refer to Fig. A-3, sheets 5 and 6 for location of testholes.

SUMMARY OF TEST HOLE DATA

Sheet 3

FIGURE A-1

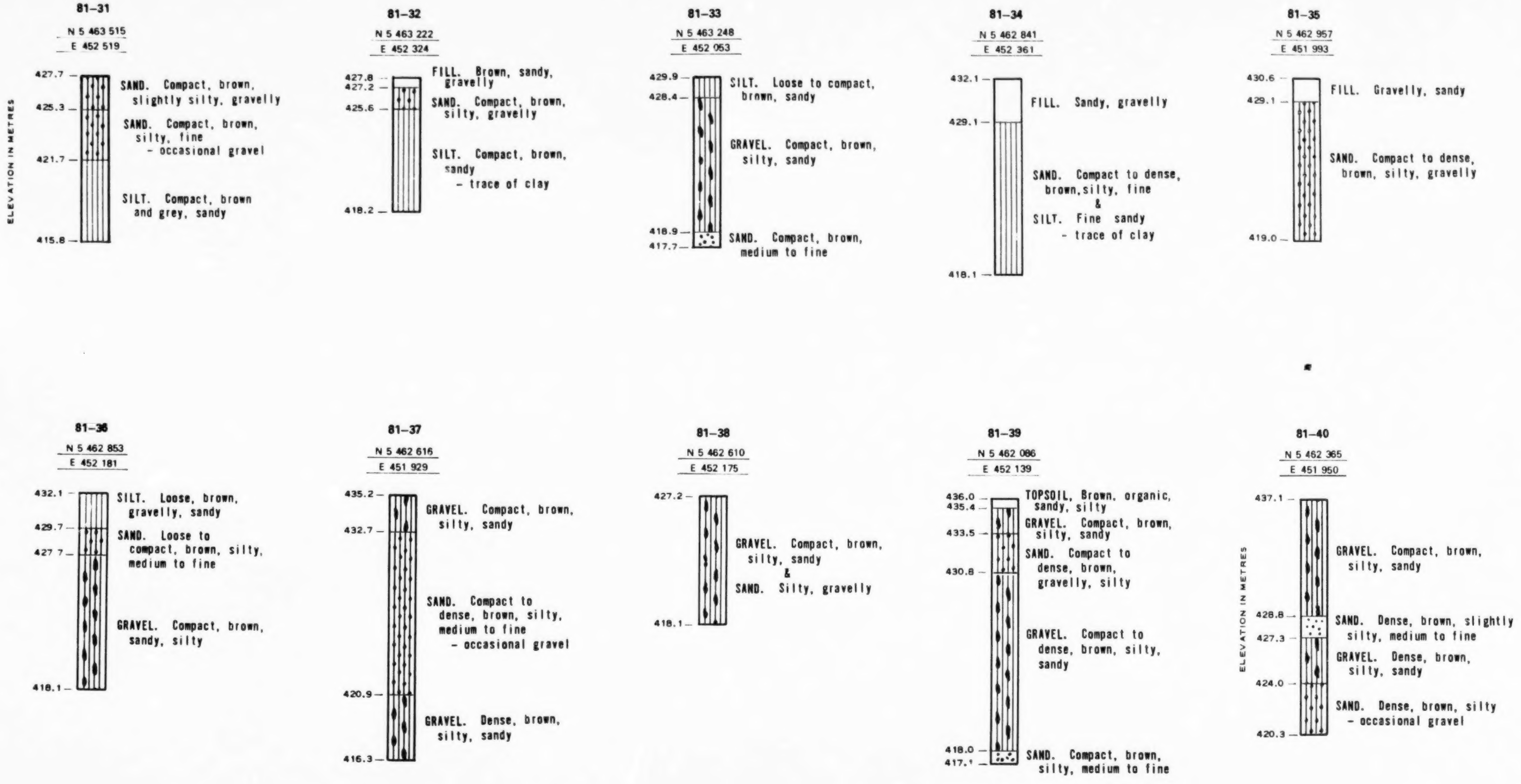

81-24
N 5 456 216
E 452 327
516.6
514.8
510.5
491.3
454.7
SAND. Compact, brown, silty, fine
SAND. Dense, brown, silty, gravelly
SAND. Compact to dense, brown, silty, medium to fine
SAND. Dense to very dense, brown, silty, fine & SILT. Fine sandy
ELEVATION IN METRES
81-25
N 5 455 643
E 452 119
507.0
504 6
476 6
471.2
456.1
SAND. Loose to compact, brown, gravelly, silty
SAND. Loose to compact, brown, silty, very fine - occasional gravel
SAND. Dense, brown, slightly silty, medium to fine
SAND. Dense to very dense, brown, silty, medium to fine
81-26
N 5 454 941
E 452 478
465.8
457.9
448.7
443.0
437.8
435.9
434.1
430.8
GRAVEL. Loose to compact, brown, silty, sandy
SAND. Compact to dense, brown, slightly gravelly, silty
GRAVEL. Dense, brown, sandy, silty
SAND. Dense, brown, slightly gravelly, silty, fine
GRAVEL. Dense, brown, silty, sandy
SAND. Dense, brown, silty, gravelly
SAND. Dense, brown, silty, medium to fine - occasional gravel
81-27
N 5 454 192
E 452 347
469.7
468.7
461.5
452.6
434.7
TOPSOIL. Loose, brown, silty
SAND. Compact, brown, silty, gravelly
SAND. Compact, brown, medium to fine - trace of silt occasional gravel
GRAVEL. Dense, brown, slightly silty, sandy
81-28
N 5 451 039
E 449 929
467.0
463.3
435.0
426.2
SAND. Compact, brown, slightly silty, gravelly
GRAVEL. Compact to dense, brown, silty, sandy
SAND. Dense, brown, silty, medium to fine
81-29
N 5 463 676
E 451 561
436.4
436.1
419.9
TOPSOIL. Brown, organic, silty, sandy
GRAVEL. Compact, brown, silty, sandy
81-30
N 5 463 728
E 452 456
428.0
423.0
414.2
413.7
412.7
ELEVATION IN METRES
GRAVEL. Loose to compact, brown, silty sandy
SAND. Loose to compact, brown and grey, silty, fine
GRAVEL. Compact, grey, sandy, silty
SAND. Compact, grey, slightly gravelly, silty

NOTE: Refer to Fig. A-3, sheets 3, 7 and 8 for location of testholes.

SUMMARY OF TEST HOLE DATA

Sheet 4

FIGURE A-1

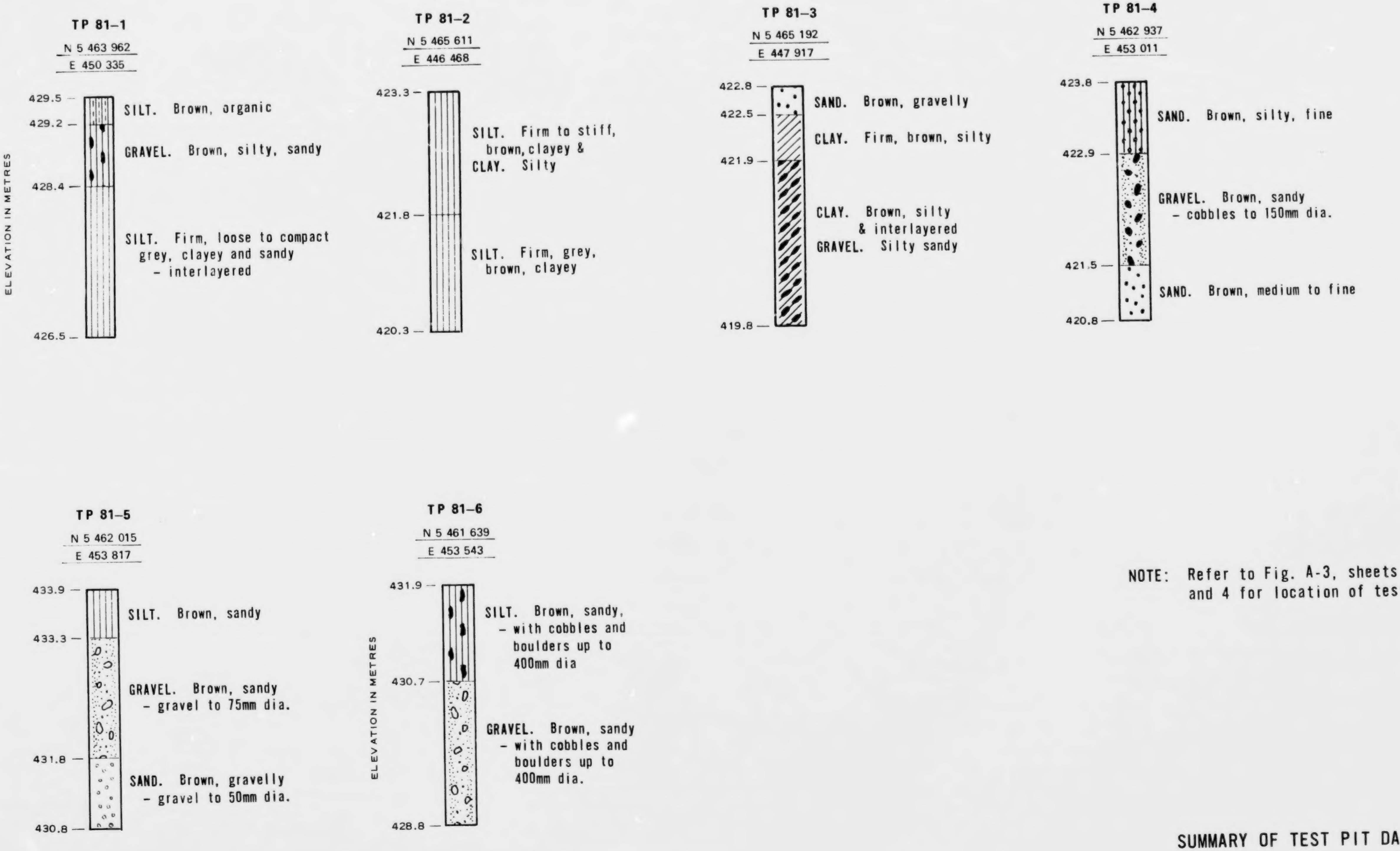

ELEVATION IN METRES
81-31
N 5 463 515
E 452 519
427.7
425.3
421.7
415.8
SAND. Compact, brown, slightly silty, gravelly
SAND. Compact, brown, silty, fine - occasional gravel
SILT. Compact, brown and grey, sandy
81-32
N 5 463 222
E 452 324
427.8
427.2
425.6
418.2
FILL. Brown, sandy, gravelly
SAND. Compact, brown, silty, gravelly
SILT. Compact, brown, sandy - trace of clay
81-33
N 5 463 248
E 452 053
429.9
428.4
418.9
417.7
SILT. Loose to compact, brown, sandy
GRAVEL. Compact, brown, silty, sandy
SAND. Compact, brown, medium to fine
81-34
N 5 462 841
E 452 361
432.1
429.1
418.1
FILL. Sandy, gravelly
SAND. Compact to dense, brown, silty, fine & SILT. Fine sandy - trace of clay
81-35
N 5 462 957
E 451 993
430.6
429.1
419.0
FILL. Gravelly, sandy
SAND. Compact to dense, brown, silty, gravelly
81-36
N 5 462 853
E 452 181
432.1
429.7
427.7
418.1
SILT. Loose, brown, gravelly, sandy
SAND. Loose to compact, brown, silty, medium to fine
GRAVEL. Compact, brown, sandy, silty
81-37
N 5 462 616
E 451 929
435.2
432.7
420.9
416.3
GRAVEL. Compact, brown, silty, sandy
SAND. Compact to dense, brown, silty, medium to fine - occasional gravel
GRAVEL. Dense, brown, silty, sandy
81-38
N 5 462 610
E 452 175
427.2
418.1
GRAVEL. Compact, brown, silty, sandy & SAND. Silty, gravelly
81-39
N 5 462 086
E 452 139
436.0
435.4
433.5
430.8
418.0
417.1
TOPSOIL. Brown, organic, sandy, silty
GRAVEL. Compact, brown, silty, sandy
SAND. Compact to dense, brown, gravelly, silty
GRAVEL. Compact to dense, brown, silty, sandy
SAND. Compact, brown, silty, medium to fine
81-40
N 5 462 365
E 451 950
ELEVATION IN METRES
437.1
428.8
427.3
424.0
420.3
GRAVEL. Compact, brown, silty, sandy
SAND. Dense, brown, slightly silty, medium to fine
GRAVEL. Dense, brown, silty, sandy
SAND. Dense, brown, silty - occasional gravel

SUMMARY OF TEST HOLE DATA

Sheet 5

**NOTE:** Refer to Fig. A-3, sheet 3 for location of testholes.

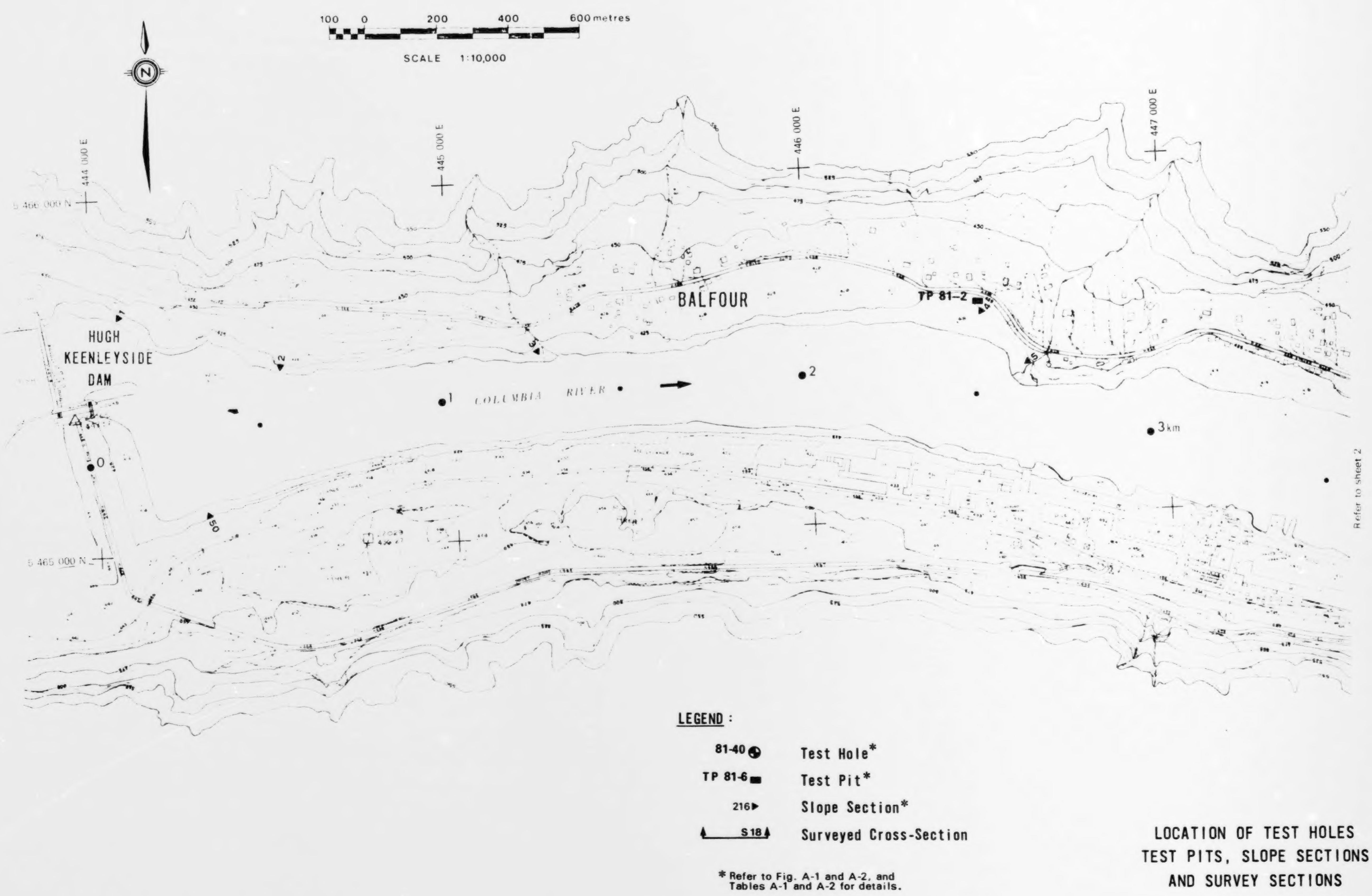
**FIGURE A-1**


TP 81–1
N 5 463 962
E 450 335
429.5
429.2
428.4
426.5
SILT. Brown, organic
GRAVEL. Brown, silty, sandy
SILT. Firm, loose to compact grey, clayey and sandy - interlayered
ELEVATION IN METRES
TP 81–2
N 5 465 611
E 446 468
423.3
421.8
420.3
SILT. Firm to stiff, brown, clayey & CLAY. Silty
SILT. Firm, grey, brown, clayey
TP 81–3
N 5 465 192
E 447 917
422.8
422.5
421.9
419.8
SAND. Brown, gravelly
CLAY. Firm, brown, silty
CLAY. Brown, silty & interlayered GRAVEL. Silty sandy
TP 81–4
N 5 462 937
E 453 011
423.8
422.9
421.5
420.8
SAND. Brown, silty, fine
GRAVEL. Brown, sandy - cobbles to 150mm dia.
SAND. Brown, medium to fine
TP 81–5
N 5 462 015
E 453 817
433.9
433.3
431.8
430.8
SILT. Brown, sandy
GRAVEL. Brown, sandy - gravel to 75mm dia.
SAND. Brown, gravelly - gravel to 50mm dia.
TP 81–6
N 5 461 639
E 453 543
431.9
430.7
428.8
ELEVATION IN METRES
SILT. Brown, sandy, - with cobbles and boulders up to 400mm dia
GRAVEL. Brown, sandy - with cobbles and boulders up to 400mm dia.

NOTE: Refer to Fig. A-3, sheets 1, 2 and 4 for location of test pits.

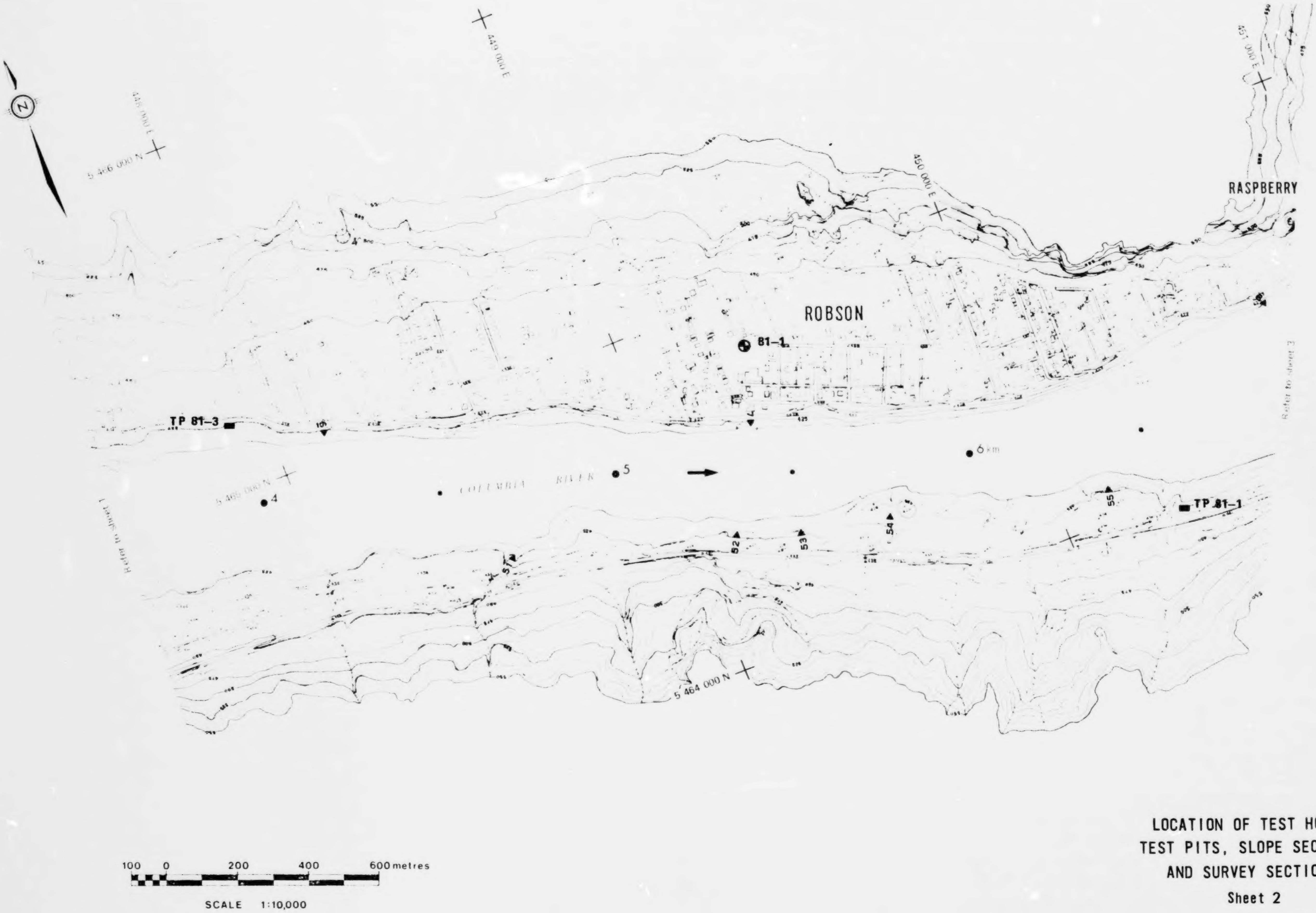
SUMMARY OF TEST PIT DATA

**FIGURE A-2**



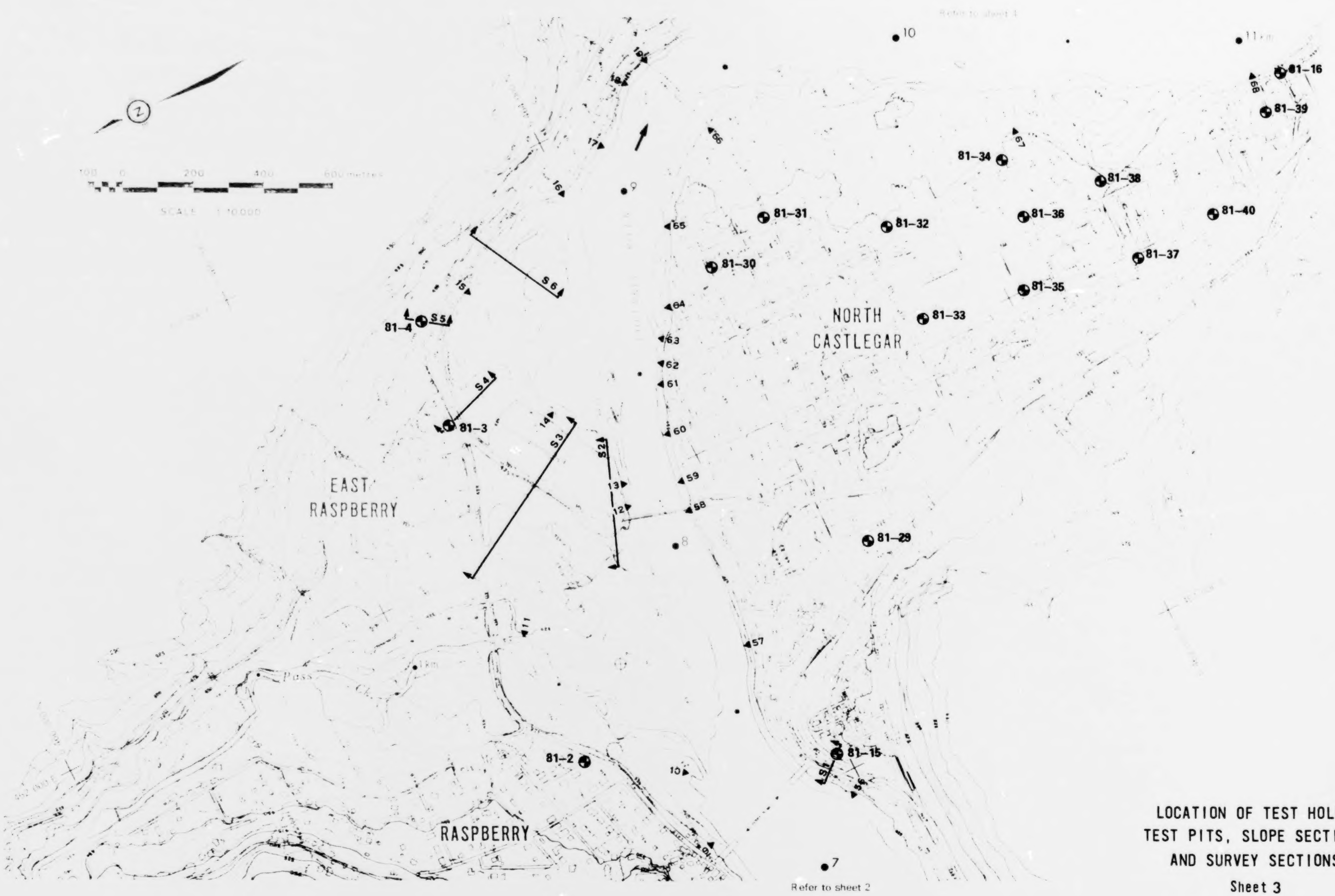
LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS
Sheet 1

FIGURE A-3



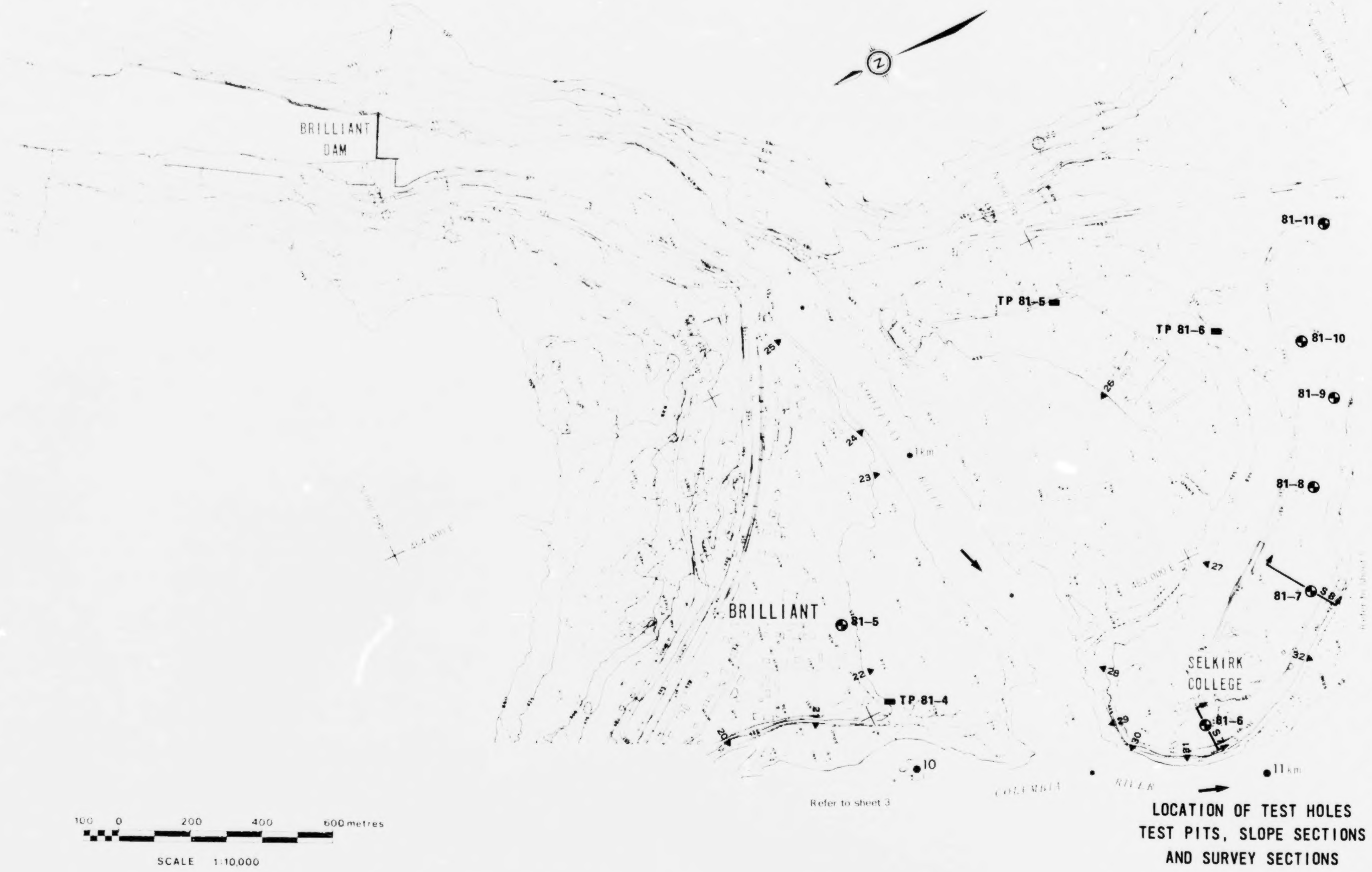
LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS
Sheet 2

FIGURE A-3



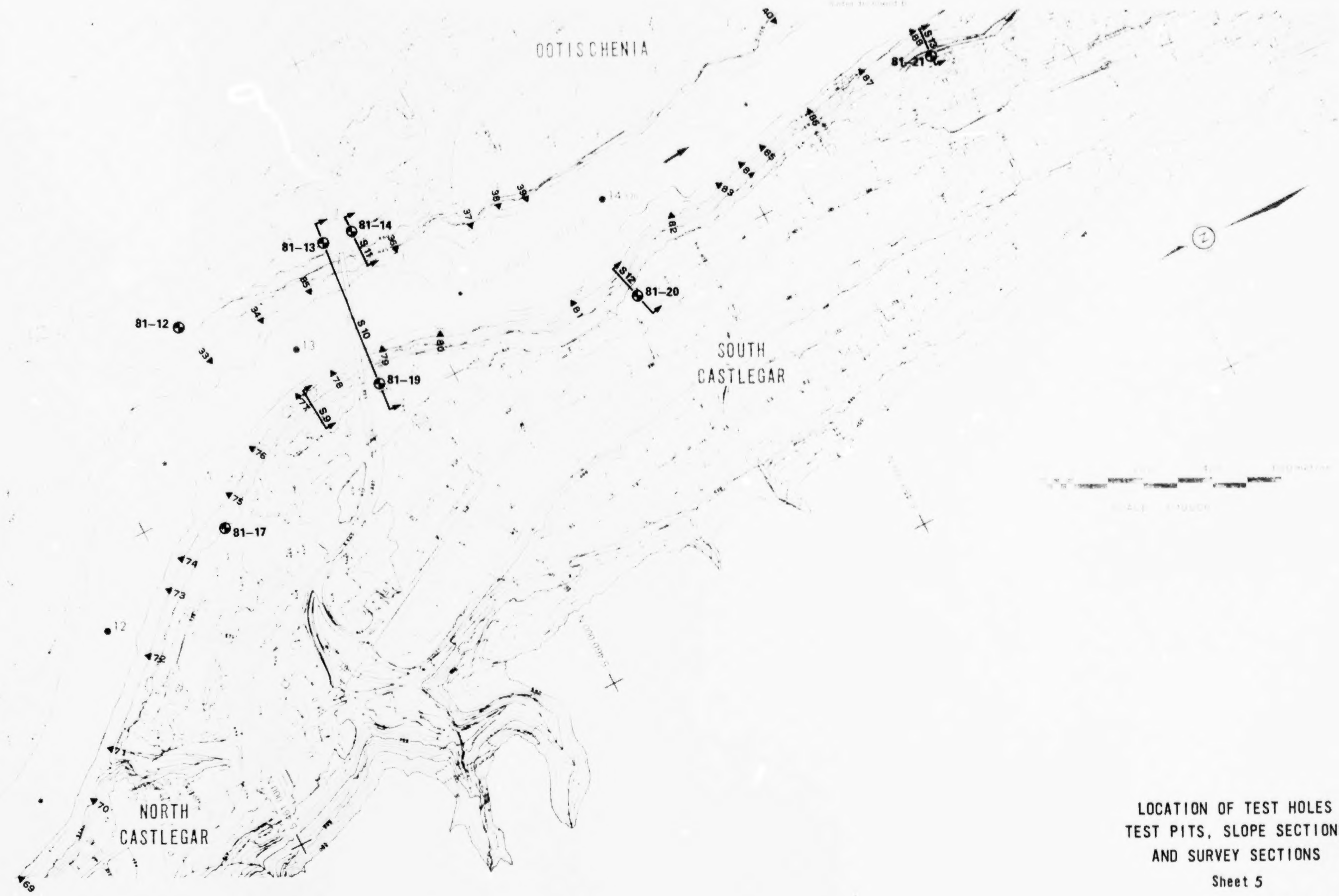
LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS
Sheet 3

FIGURE A-3



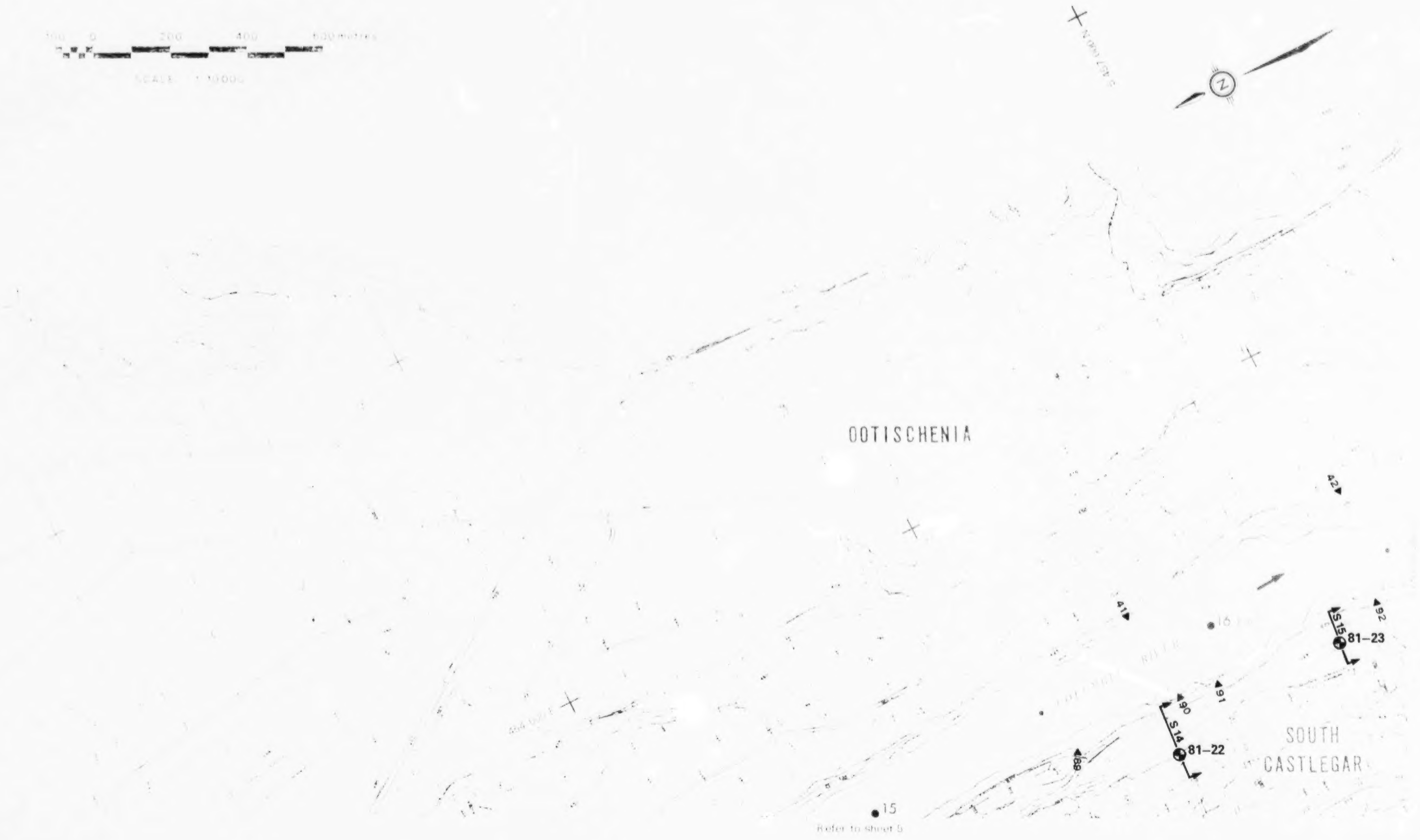
LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS
Sheet 4

FIGURE A-3



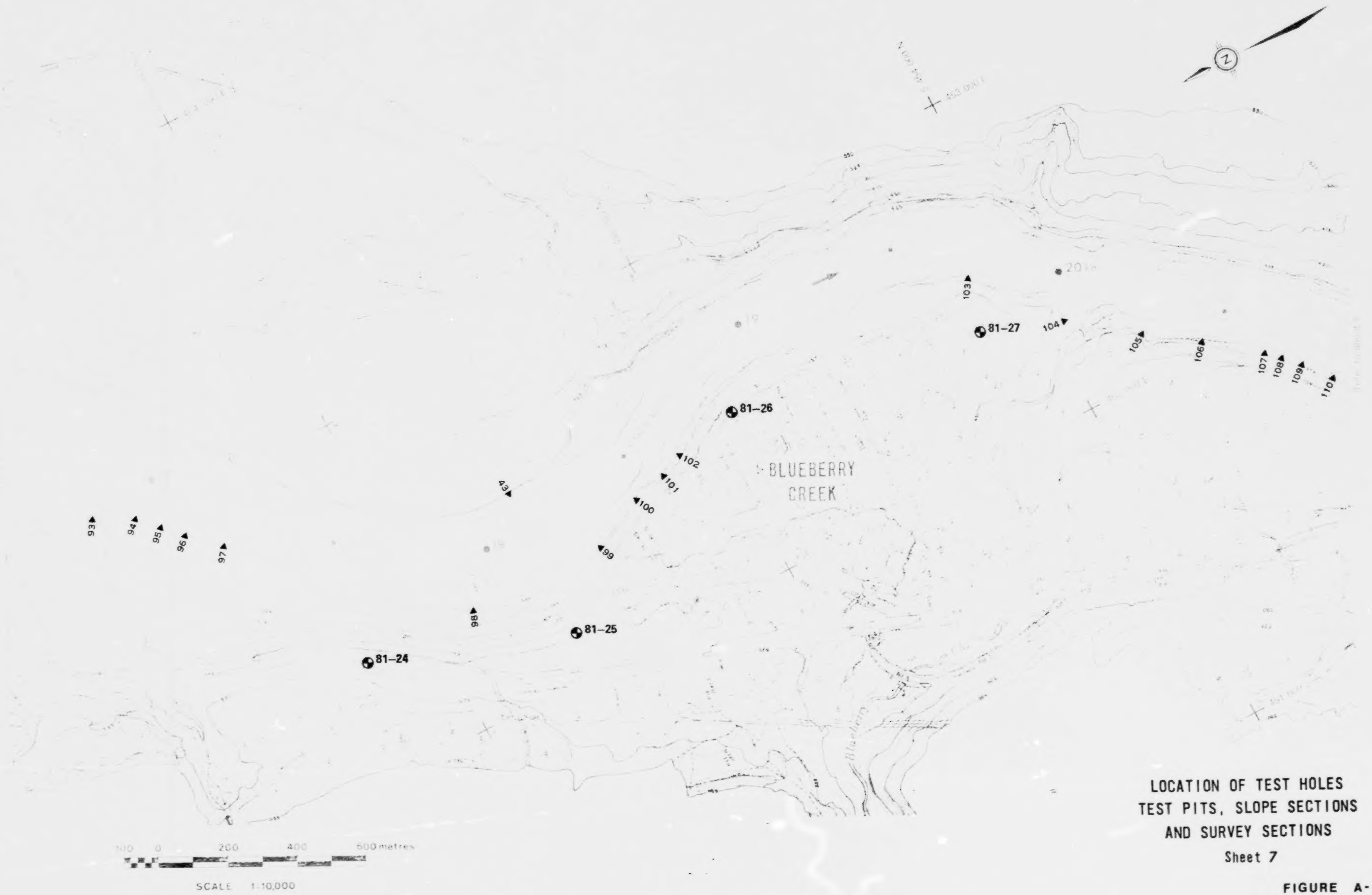
LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS
Sheet 5

FIGURE A-3



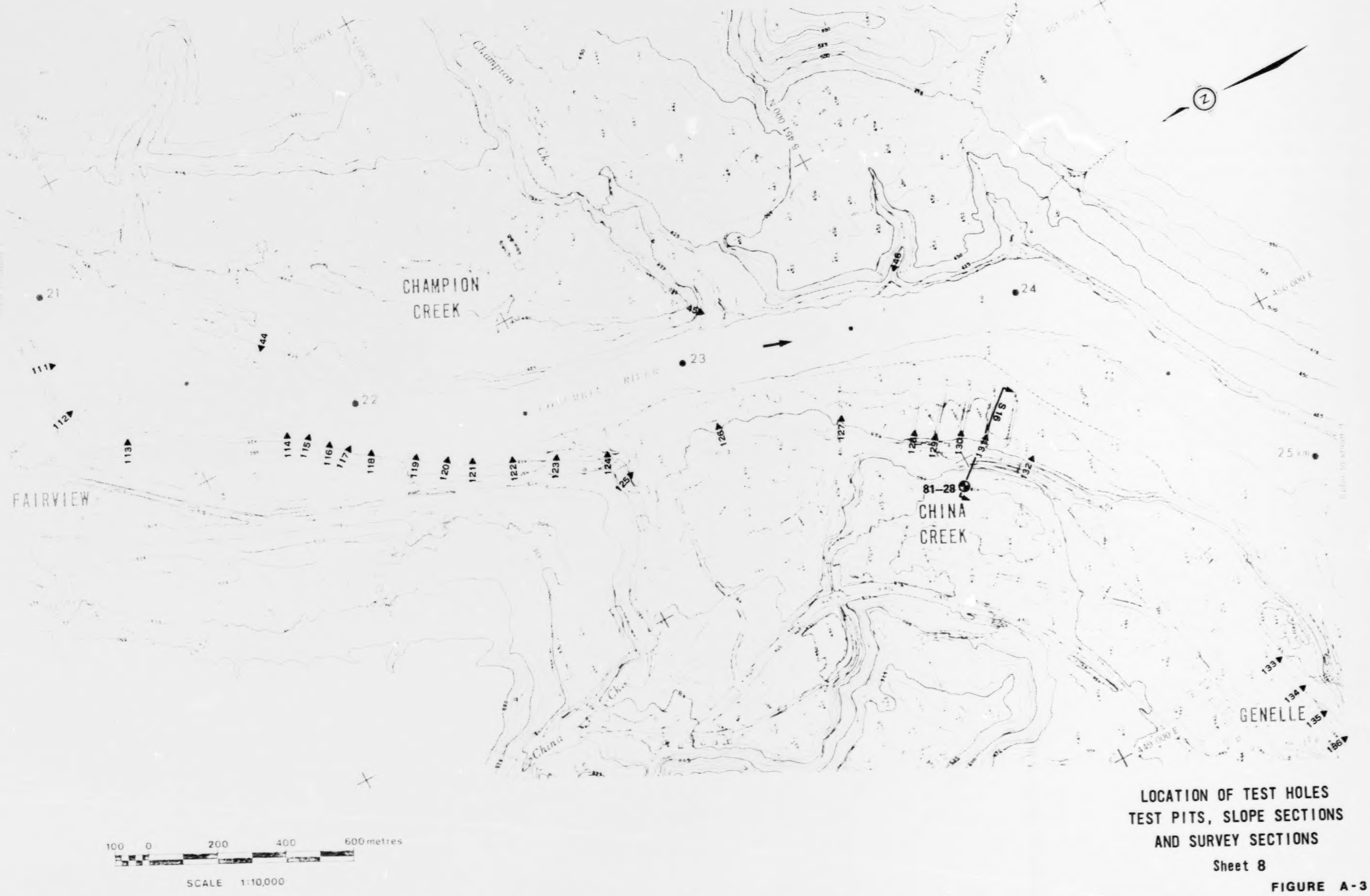
LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS
Sheet 6

**FIGURE A-3**



LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS
Sheet 7

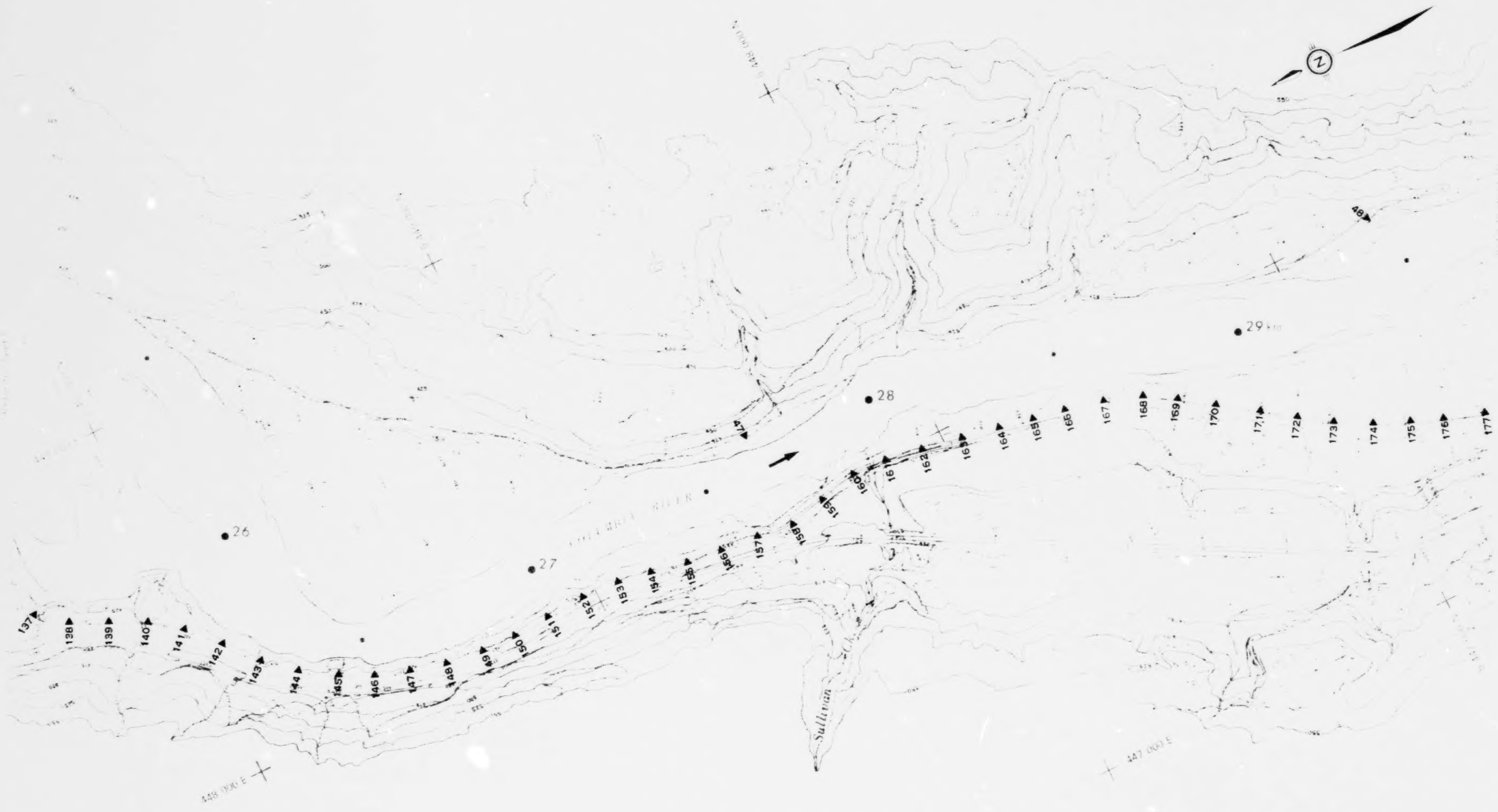
FIGURE A-3



LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS

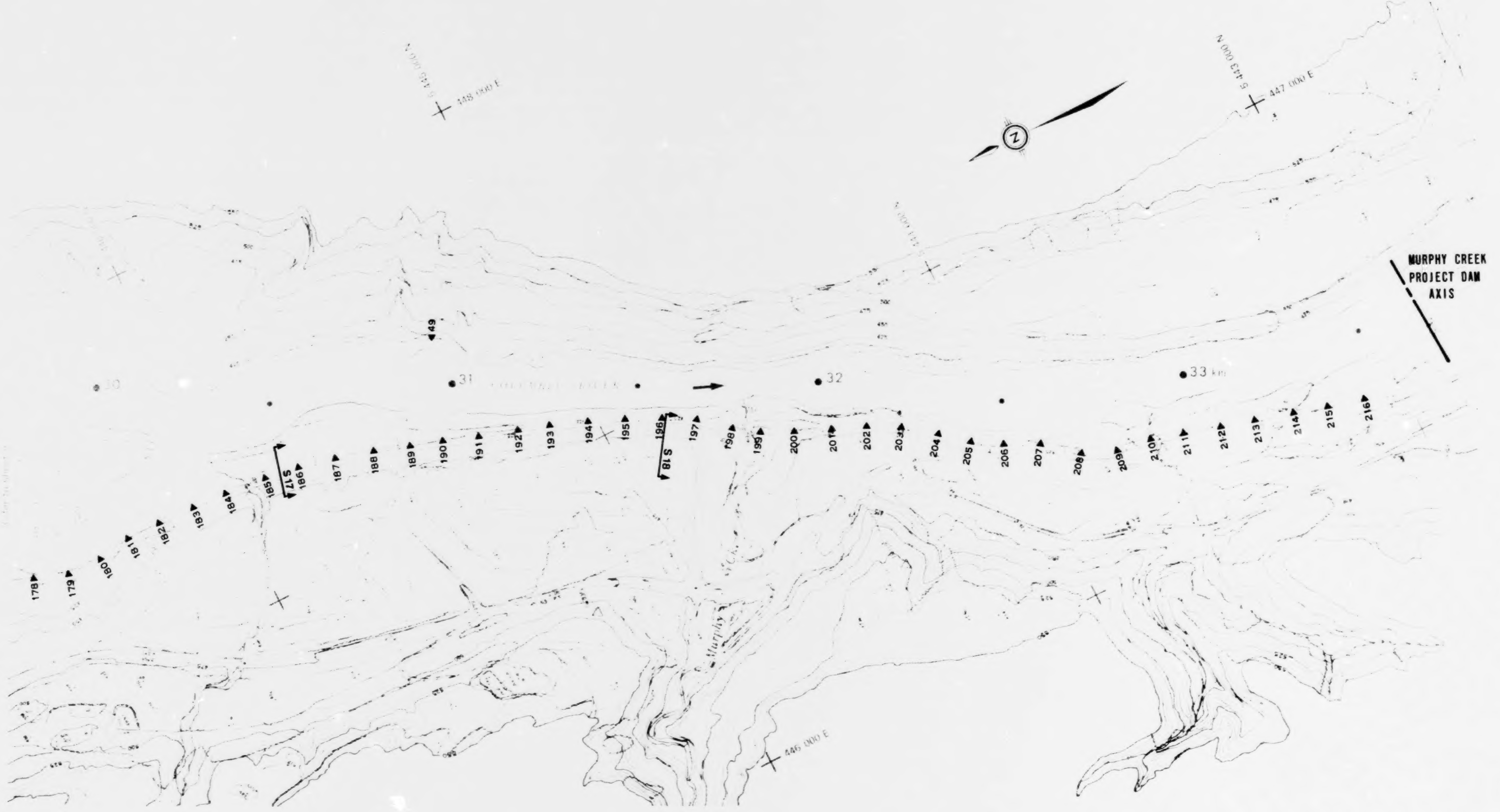
Sheet 8

**FIGURE A-3**



LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS
Sheet 9

FIGURE A-3



LOCATION OF TEST HOLES
TEST PITS, SLOPE SECTIONS
AND SURVEY SECTIONS
Sheet 10

**FIGURE A-3**

# APPENDIX B

## DISCUSSION OF HYDRAULIC ASPECTS

B.1 Duration of Wave Generating Winds
B.2 Selection of Freeboard and Design Wave Conditions
B.3 Protection against Current Erosion

## B.1 DURATION OF WAVE GENERATING WINDS

The main mechanism of reservoir bank erosion is progressive beaching of slopes due to wave action. The intensity and duration of wave action is an important factor in estimating the amount of bank regression with time.

Hourly wind records obtained at Castlegar Airport for a randomly selected year (1967) were assumed to be representative of winds in the reservoir area and were used to compute the duration of wave-generating winds in the reservoir area during the year. These wave-generating winds were categorized as follows:

Type A - 15 to 20 km/hr

Type B - 20 to 30 km/hr

Type C - more than 30 km/hr

For an average fetch of 800 m (refer Section B.2), the significant wave heights for the above wind categories are as follows:

Type A - up to 0.25 m reached within approx. 30 min.

Type B - 0.25 to 0.35 m reached within 20 to 30 min.

Type C - 0.6 m for winds of 80 km/hr (over reservoir) blowing for 15 min., and 0.7 m for winds of 120 km/hr blowing for 10 min.

The duration of the wave generating winds throughout the year are plotted as a histogram on Fig. B-1. From this plot it is evident that wave generating winds blew for 11 percent of the year and that the durations of each wind category (expressed as a proportion of the total period of wave generating winds for the year) were:

Type A - 40 percent

Type B - 56 percent

Type C - 4 percent

Fig. B-2 is a histogram showing the duration (for a 12-month period) of wave generating winds at various reservoir levels, based on the mean forebay levels shown on Fig. 3-2 and the wind duration data shown on Fig. B-1.

Fig. B-2 clearly indicates that most of the wave generating wind activity occurs when the reservoir is between el 423 and 424 m. Therefore, the range over which the beach slope will form includes this elevation range. When the reservoir level is between 419 and 423 m, only a relatively small amount of wave generating wind activity occurs and it is unlikely, considering the nature of the bank materials, that any appreciable beaching erosion at these lower levels will occur during the expected life of the project. Fig. B-2 indicates a significant amount of wave generating wind activity while the reservoir is below el 419 m, however it is expected that redeposition of material eroded from higher levels will heal beaching at these lower levels.

## B.2 SELECTION OF FREEBOARD AND DESIGN WAVE CONDITIONS

Design wave conditions for a particular section of reservoir shoreline depend primarily on the assumed maximum wind conditions and on the reservoir fetch. Six years of wind records from Castlegar Airport were analyzed. They show preferred wind directions up and down the valley, with a maximum observed hourly wind speed of 72 km/hr. Allowing for the shortness of the record used, and for the fact that on the narrow and winding Murphy Creek Reservoir the maximum wave height would be attained in a matter of 10 to 20 minutes, a design wind speed of up to 120 km/hr was conservatively selected. With the relatively small fetch of the proposed reservoir, winds in excess of the design speed would not result in an appreciable increase in wave height.

Fetch can be determined in various ways. Often it is simply measured off maps as the longest straight line over water visible from any particular point along the shore. In the case of a narrow river-like reservoir such as Murphy Creek, this leads to gross overestimates since waves reach much greater height on wide open water than in a narrow confined channel with an irregular shoreline. Following the procedure given in Davis and Sorenson, 1969, which in turn is based on a U.S. Corps of Engineers study, one can determine an effective fetch which averages the fetch over a 90° angle. In the case of Murphy Creek, the computed effective fetch ranges from a few hundred meters to roughly 1.0 km. For the purpose of this preliminary study an average fetch of 800 m has been adopted. The significant wave height corresponding to a wind of 120 km/hr over a fetch of 800 m is 0.7 m. To generate this condition the wind has to blow for more than 10 minutes.

The effects of waves on a shoreline depends on the angle at which the waves strike the shore, and on the roughness, slope angle and permeability of the shore. Along Murphy Creek Reservoir there are only a few sites where significant waves would strike at angles close to 90° since most of the shores are parallel to the predominant wave directions. Wave run up, the maximum height above the static reservoir level reached by waves, can be determined from a set of diagrams in Davis and Sorensen, 1969 (p. 4-20). Two types of slopes are of primary interest here, the relatively flat and smooth beach slopes that are likely to develop in sand and fine gravel at unprotected sites, and the much steeper and coarser slopes of riprap protected banks or naturally coarse, stable banks. Fig. B-3 shows the range of wave run up to be expected for conditions in the Murphy Creek Reservoir. The figure is based on a maximum wave height of 1.15 m, which corresponds to the above significant wave height of 0.7 m and includes an allowance of 0.05 m for wind setup. The curves should only be considered as rough guidelines, but they do provide a basis for the selection of appropriate freeboard values. For roads, railways, and other industrial facilities, 1.5 m is suggested in areas of maximum exposure, while a more conservative 2.0 m should be used for residences.

## B.3 PROTECTION AGAINST CURRENT EROSION

Erosion of non-cohesive materials by flowing water depends on the balance between the forces trying to displace individual particles and the forces associated with the particle's ability to withstand movement. The moving forces of shear and lift vary greatly in time and space due to the effects of turbulence and channel irregularities, but are closely related to the average bed shear, $\tau_{avg}$, which can be computed as follows:

$$\tau_{avg} = g\rho RS$$

where $g$ = Acceleration of gravity = 9.82 $m/s^2$
$\rho$ = Density of water = 1000 $kg/m^3$
$R$ = Hydraulic radius
$S$ = Slope of the energy grade line

For the purpose of calculations, $\tau_{avg}$ can be replaced by a mean channel velocity and a measure of channel size, since the boundary shear also determines the velocity gradient normal to the boundary.

The resisting forces depend primarily on particle size, D; the submerged specific weight of the bed material particles, $\rho_s$; the particle shape and exposure; and in the case of river banks on the bank slope angle, $\alpha$. Assuming that the stable grain size for a given flow condition is the main item of interest, this can be expressed either in terms of shear, as

$$D = f(\tau_{avg}, \rho_s, \alpha, Ki)$$

where Ki represents secondary factors such as particule shape, particle exposure, flow curvature, etc.; or in terms of velocity as

$$D = g(V_{avg}, R, \rho_s, \alpha, Ki)$$

in which $V_{avg}$ is the average channel velocity.

The above two functions have been studied extensively and results are available in design charts. The analysis used for this report is based primarily on Anderson et al. 1970, and Neill, 1968.

Along the reach of interest, the Columbia River valley is a typical, highly stable lake outlet channel that has been formed by degradation of the post glacial river through various types of glacial sediments. Upstream of the Kootenay River confluence the Columbia River has no supply of coarse sediments other than its own bed and banks, and downstream of the confluence the supply was small prior to regulation of the river, and is now practically zero. This lack of coarse material in the load of the river indicates that the Columbia was, for the most part, stable, even under large unregulated flood conditions. An exception to this condition is the section on the right bank from km 10.8 to 11.6 and km 26.2 to 26.5 downstream from China Creek. With present floods lowered by upstream regulation it is believed that the river channel is essentially stable.

The Murphy Creek Dam will affect current erosion in two ways by:

(i) Raising water levels, and
(ii) Reducing the energy line slope.

The combined effect is a reduction in the erosive capacity of the the flow, with the amount of reduction decreasing with distance from the dam. Even so it is considered that the above mentioned right bank section from km 10.8 to 11.6 is still insufficiently armoured to withstand flows during reservoir flood conditions.

As shown on Fig. 3-1, the raising of the river levels brings water in contact with areas of the river banks that previously have not been reached by the river current, specifically from km 22.8 on the right bank and 23.1 on the left bank downstream to the damsite. Therefore, in this portion of the reservoir there is a potential for erosion. This current erosion is not significant, however, for when the reservoir levels are high the erosive capacity of the current is minimal. The erosive velocites occur during periods of drawdown due to flood conditions, specifically, during the spillway design flood (regulated 200 year flood), at which time the water level is below the profile for the natural 50 year flood. Over the entire length of the reservoir, therefore, erosive currents will act only on areas previously exposed to flows at least equal in severity to the spillway design flood.

As a guide to the design of bank protection, two grain sizes have been computed for each of the river cross sections used in the backwater computations.* The first value, $D_1$, is the stable grain size for the central portion of the channel under 200 year flood conditions with Murphy Creek Dam in place (regulated flood). It is stated for interest only as the general stability of the river channel is assured by geologic setting and reduced floods due to upsteam regulation. The second value, $D_2$, refers to the stable grain size required for stability of the upper portion of the river bank. It is based on an assumed depth of 3 m below full supply level, which covers the bank areas where erosion could possibly be a design consideration. In exposed areas where the main current impinges on the bank, the basic $D_2$ size needs to be multiplied by a factor of 2.5. The bank slope angle has been assumed to be flatter than 2 horizontal to 1 vertical as the angle becomes significant with steeper slopes. These grain sizes are summarized in Table B1-1.

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*B.C. Hydro HEC 2 computer program, September 1981.

The computed $D_2$ is highly variable from section to section, but this variability is probably not fully representative of natural conditions, in which more consistent grain sizes are observed. For design purposes the most critical of the computed $D_2$ values has been applied to morphologically uniform reaches such as the outside of a bend.

TABLE B-1

PARTICLE SIZE REQUIREMENTS TO MITIGATE CURRENT EROSION

| B.C. HYDRO SECTION NO.*1 | LOCATION km *2 | CALCULATED STABLE GRAIN SIZE(mm)*3 | | ADOPTED DESIGN CRITERIA *4 | | | |
|---|---|---|---|---|---|---|---|
| | | $D_1$ CENTRAL PORTION OF CHANNEL | $D_2$ SHORE 3 m BELOW FSL | LEFT SHORE | | RIGHT SHORE | |
| | | | | LOCATION km *2 | MATERIAL SIZE (mm) | LOCATION km *2 | MATERIAL SIZE (mm) |
| 0.00 | 0 | 1.07 | 0.24 | | | | |
| 1.00 | 1.08 | 1.07 | 0.24 | | | | |
| 2.00 | 2.06 | 0.35 | 0.06 | | | | |
| 3.00 | 2.68 | 0.79 | 0.18 | | | | |
| 4.00 | 3.56 | 0.27 | 0.06 | | | | |
| 5.00 | 5.42 | 0.34 | 0.06 | | | | |
| 6.00 | 6.45 | 0.31 | 0.06 | | | | |
| 6.50 | 7.22 | 0.51 | 0.12 | | | | |
| 7.00 | 7.88 | 15. | 12.6 | | | 7.4 ↓ | 75 ↓ |
| 7.40 | 8.12 | 16. | 13. | 7.9 ↓ | 75 ↓ | | |
| 7.60 | 8.15 | 87. | 20. | | | | |
| 7.80 | 8.18 | 87. | 21. | | | | |
| 8.00 | 8.30 | 88. | 21. | | | | |
| 9.00 | 8.49 | 11. | 5.0 | 8.4 | 75 | | |
| 10.00 | 8.88 | 47. | 18. | | | | |
| 11.00 | 9.56 | 0.42 | 0.24 | | | 8.5 | 75 |
| 12.00 | 10.14 | 38. | 21. | | | | |
| | | | | 10.5 ↓ | 50 ↓ | | |
| 13.00 | 11.30 | 135. | 37. | | | 10.8 ↓ | 150 ↓ |
| 17.00 | 11.98 | 33. | 14. | | | 11.6 ↓ | 100 ↓ |
| | | | | 12.4 ↓ | 75 ↓ | | |
| 18.00 | 12.85 | 135. | 34. | | | 12.5 ↓ | 150 ↓ |
| 18.40 | 13.12 | 140. | 36. | | | | |
| 18.60 | 13.15 | 81. | 14. | | | | |
| 18.80 | 13.19 | 81. | 15. | | | | |
| 19.00 | 13.33 | 91. | 16. | | | 13.2 ↓ | 75 ↓ |
| 20.00 | 14.61 | 518. | 124. | | | 13.8 ↓ | 150 ↓ |
| 20.10 | 15.42 | 9.1 | 2.3 | | | 15.0 ↓ | 75 ↓ |
| 21.00 | 16.23 | 36. | 10.3 | | | | |
| | | | | 17.2 ↓ | 50 ↓ | 16.5 ↓ | 50 ↓ |
| 22.00 | 18.05 | 68. | 17. | | | 17.7 ↓ | 75 ↓ |
| 23.00 | 19.17 | 176. | 41. | 18.3 ↓ | 75 ↓ | | |
| 24.00 | 20.74 | 52. | 13. | 20.4 ↓ | 50 ↓ | 19.9 ↓ | 50 ↓ |
| 25.00 | 22.05 | 42. | 9.3 | | | | |
| 26.00 | 24.64 | 172. | 71. | 23.1 ↕ | 50 ↕ | 22.7 ↕ | 50 ↕ |
| 27.00 | 25.70 | 21. | 6.2 | | | | |
| 27.10 | 27.17 | | | | | | |
| 27.20 | 28.84 | 28. | 7.7 | | | | |
| 28.00 | 30.00 | 16. | 5.9 | | CURRENT EROSION NOT A DESIGN CONSIDERATION | | |
| 28.10 | 31.67 | 96. | 26. | | | | |
| 28.20 | 33.65 | 80. | 18. | | | | |

Notes:

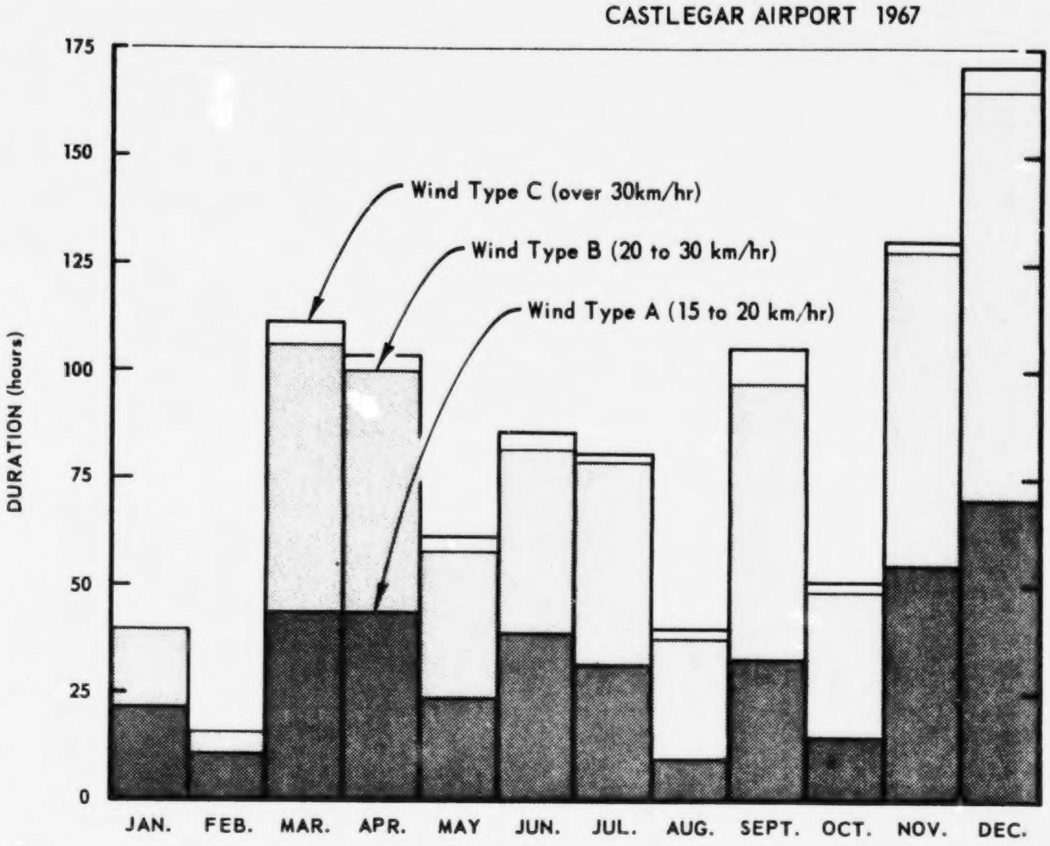
*1 Hydraulic Section from B.C. Hydro HEC II computer run dated November 26, 1981.

*2 Location refers to distance downstream from Hugh Keenlyside Dam.

*3 Stable grain sizes calculated from information provided in November 26, 1981 HEC II run, assuming a trapezoidal section, and no exposure considerations.

*4 Calculated stable grain size with a suitable factor applied for exposure conditions. Where no information is provided in this column, exposure is not of concern.

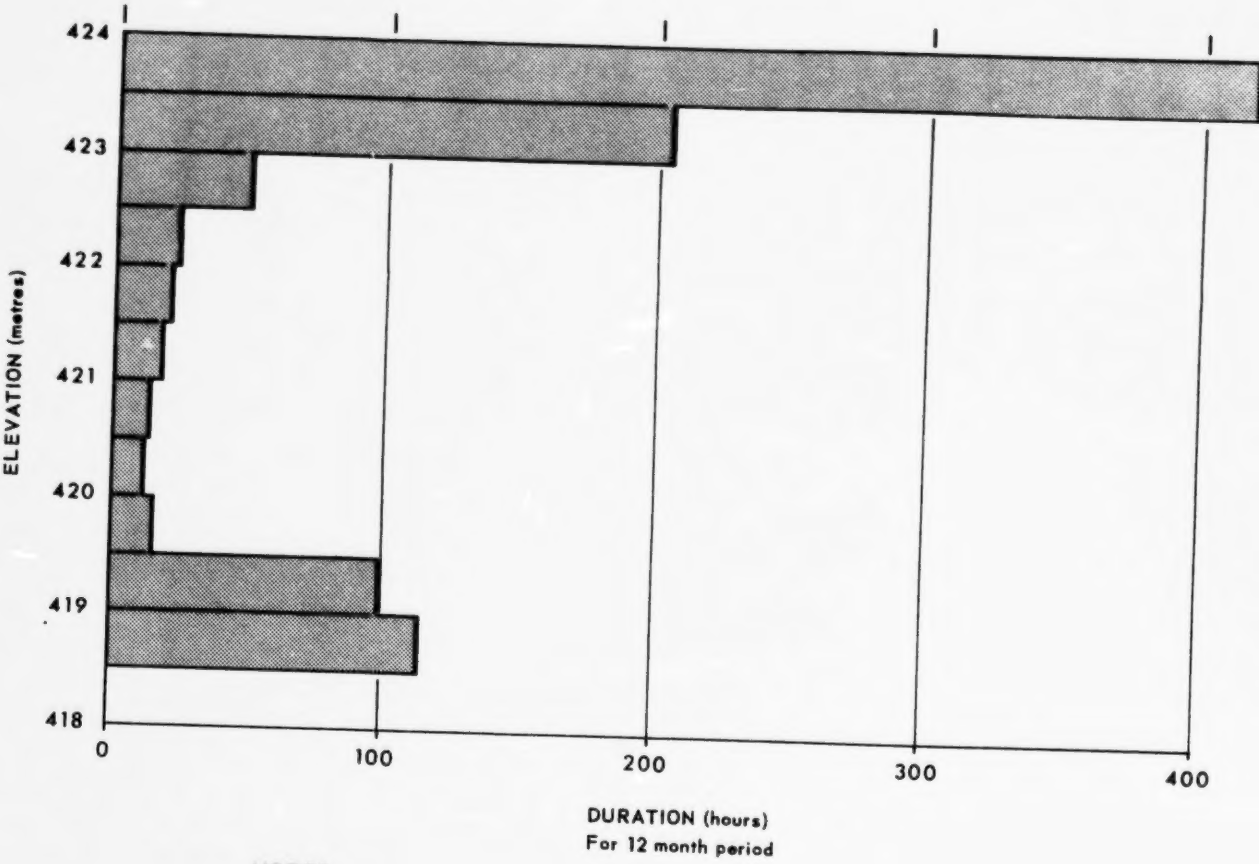
Note that particle sizes shown refer to 2 on 1 slopes or flatter.



DURATION OF WAVE GENERATING WINDS
IN RESERVOIR AREA

THURBER

FIGURE B-1



NOTES:

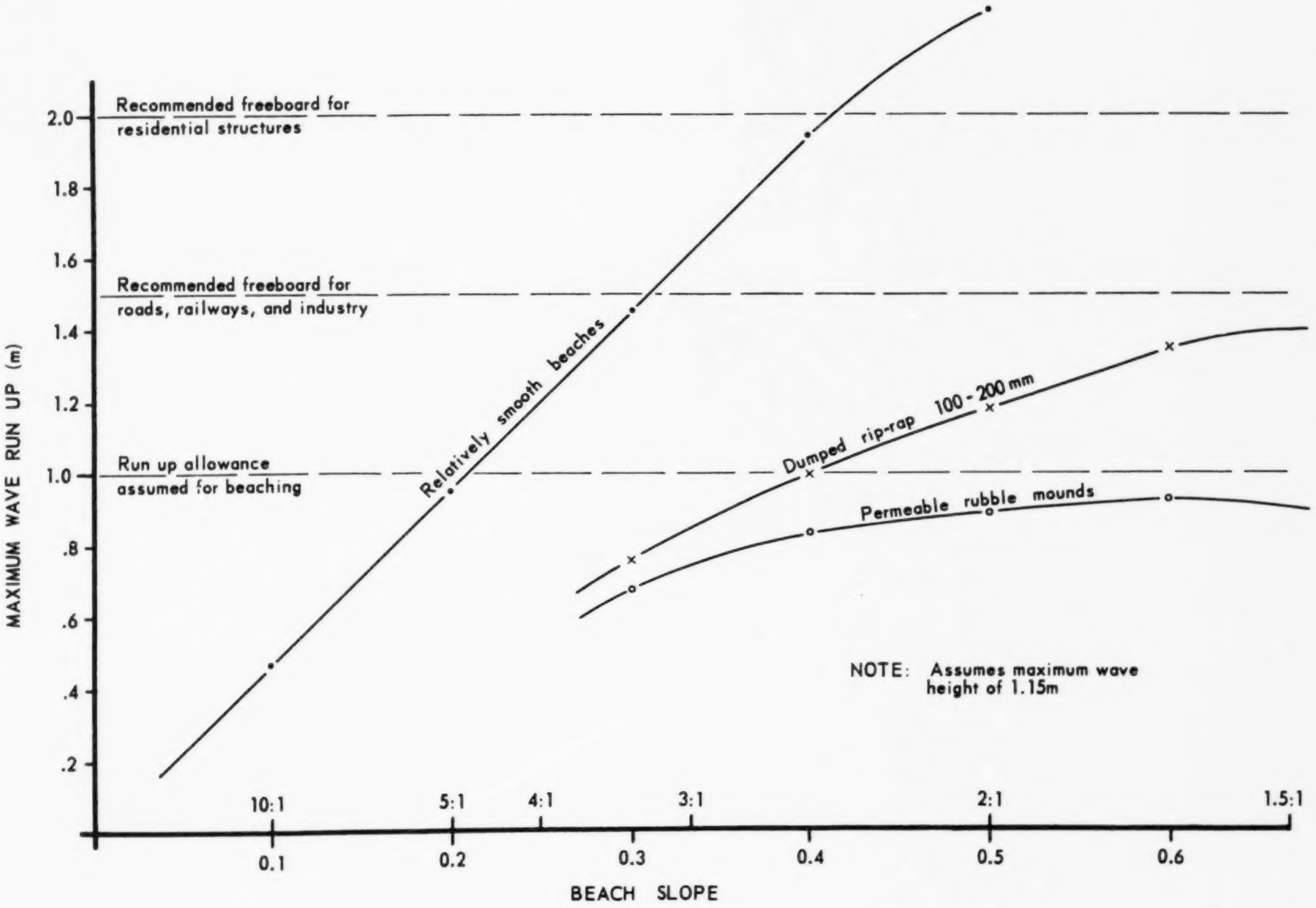
1. Plot is based on mean reservoir level (Figure 3-2), and average monthly duration of wave generating winds recorded at Castlegar Airport (Figure B-1).

2. Wave generating winds are defined as winds with a speed of at least 15km/hr blowing for up to 30 mins. For a stretch of 1km, these winds would create significant wave heights in excess of 0.25m. For 120km/hr, the significant wave height would be 0.7m.

ESTIMATED DURATION OF WAVE GENERATING WINDS
vs RESERVOIR LEVEL

THURBER

FIGURE B-2

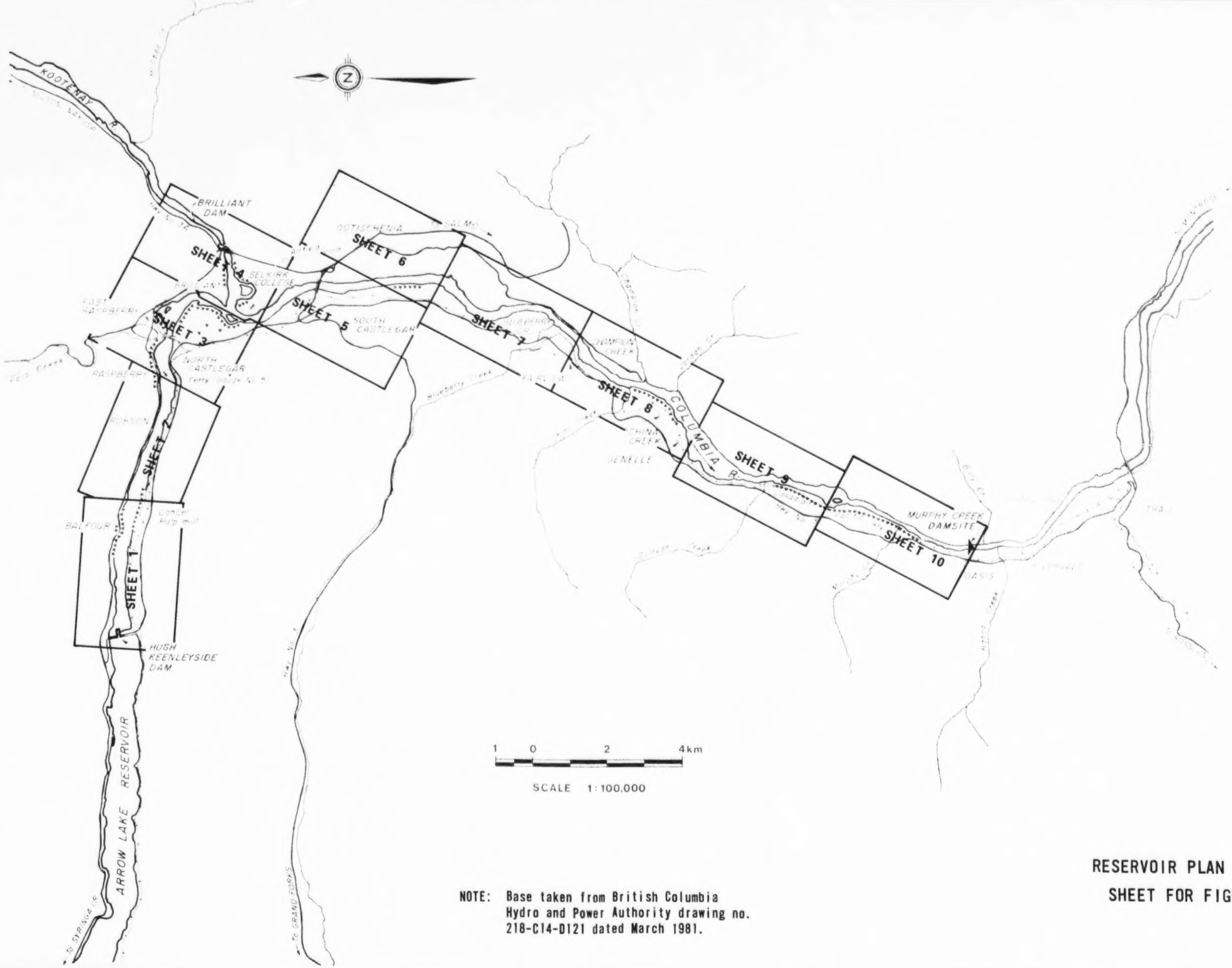


DESIGN WAVE RUN-UP ALONG
MURPHY CREEK RESERVOIR

FIGURE B-3

# APPENDIX C

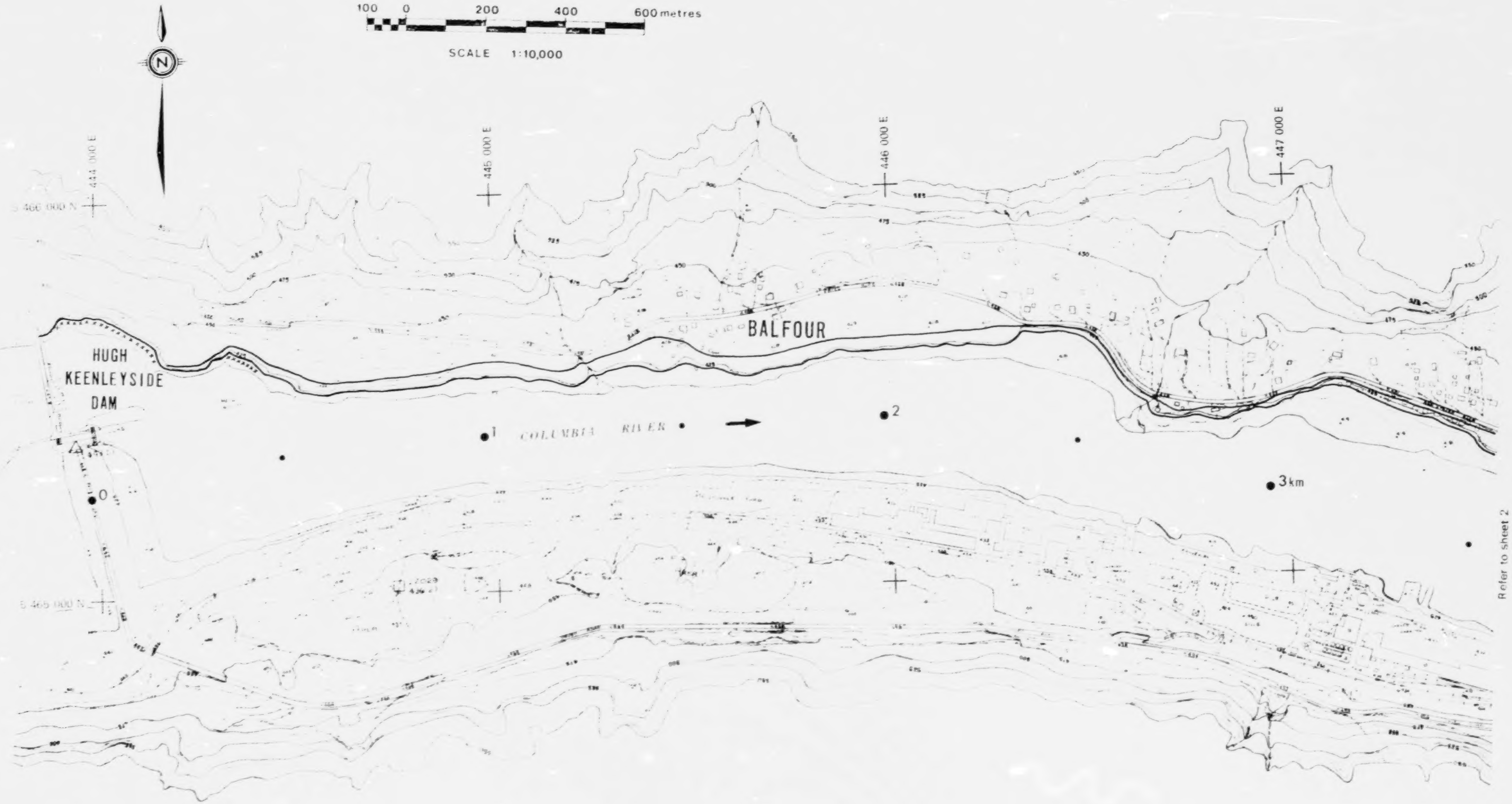
## SAFELINE DRAWINGS



NOTE: Base taken from British Columbia Hydro and Power Authority drawing no. 218-C14-D121 dated March 1981.

RESERVOIR PLAN AND INDEX SHEET FOR FIGURE C-2

FIGURE C-1



**LEGEND**

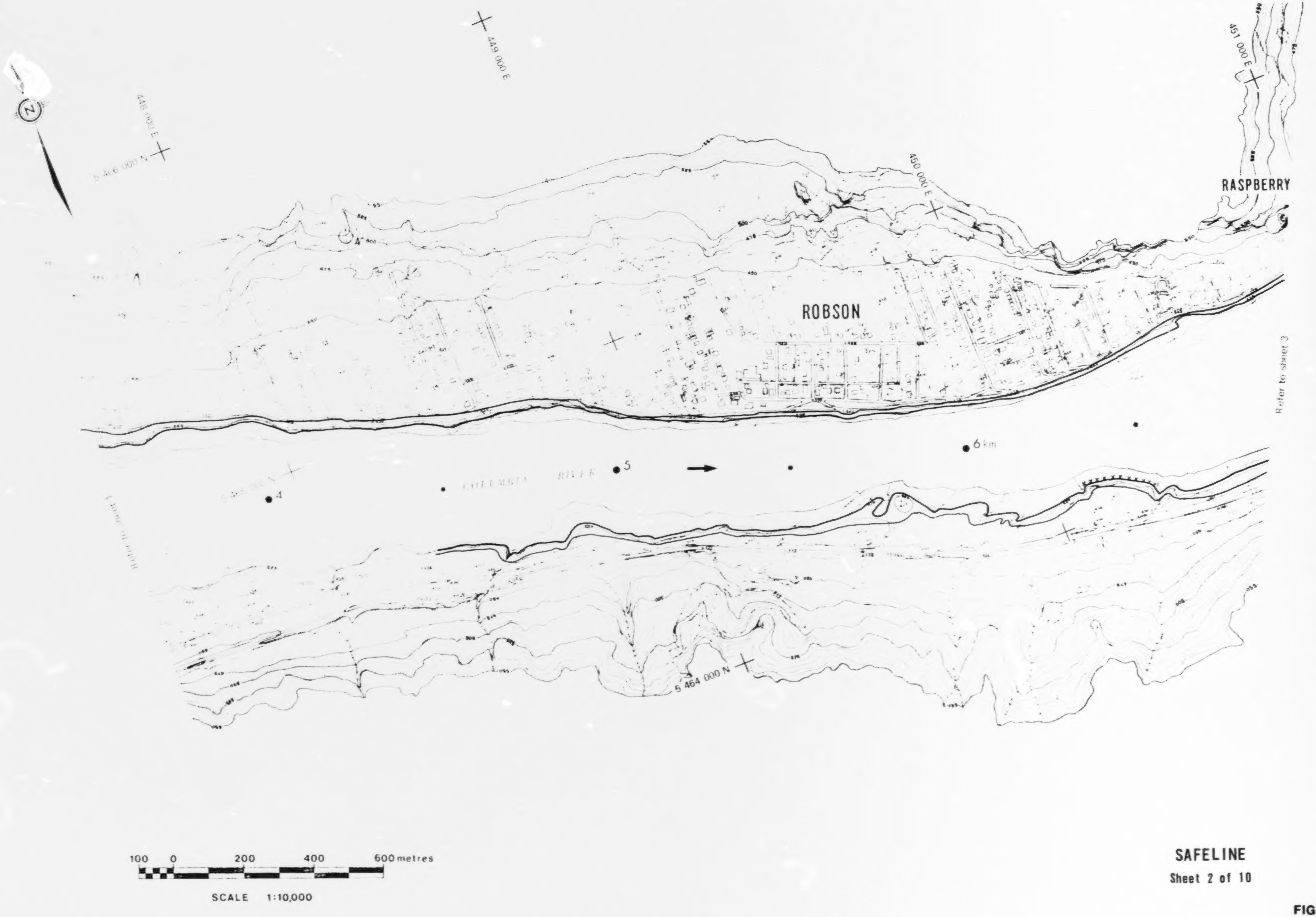
| | |
|---|---|
| ——— | Safeline without protection* |
| ═══ | Full supply level (424 m) |
| ×××××××× | Areas of existing oversteep slopes |
| ⊢— —⊣ | Areas where minimum freeboard allowances have been reduced (see sheets 3, 4 & 9) |

* Refer to Tables 4-1 and 4-2 for comments relating to the safeline, and safeline offsets at specific locations.

**SAFELINE**

**Sheet 1 of 10**

**FIGURE C-2**



**SAFELINE**

Sheet 2 of 10

**FIGURE C-2**

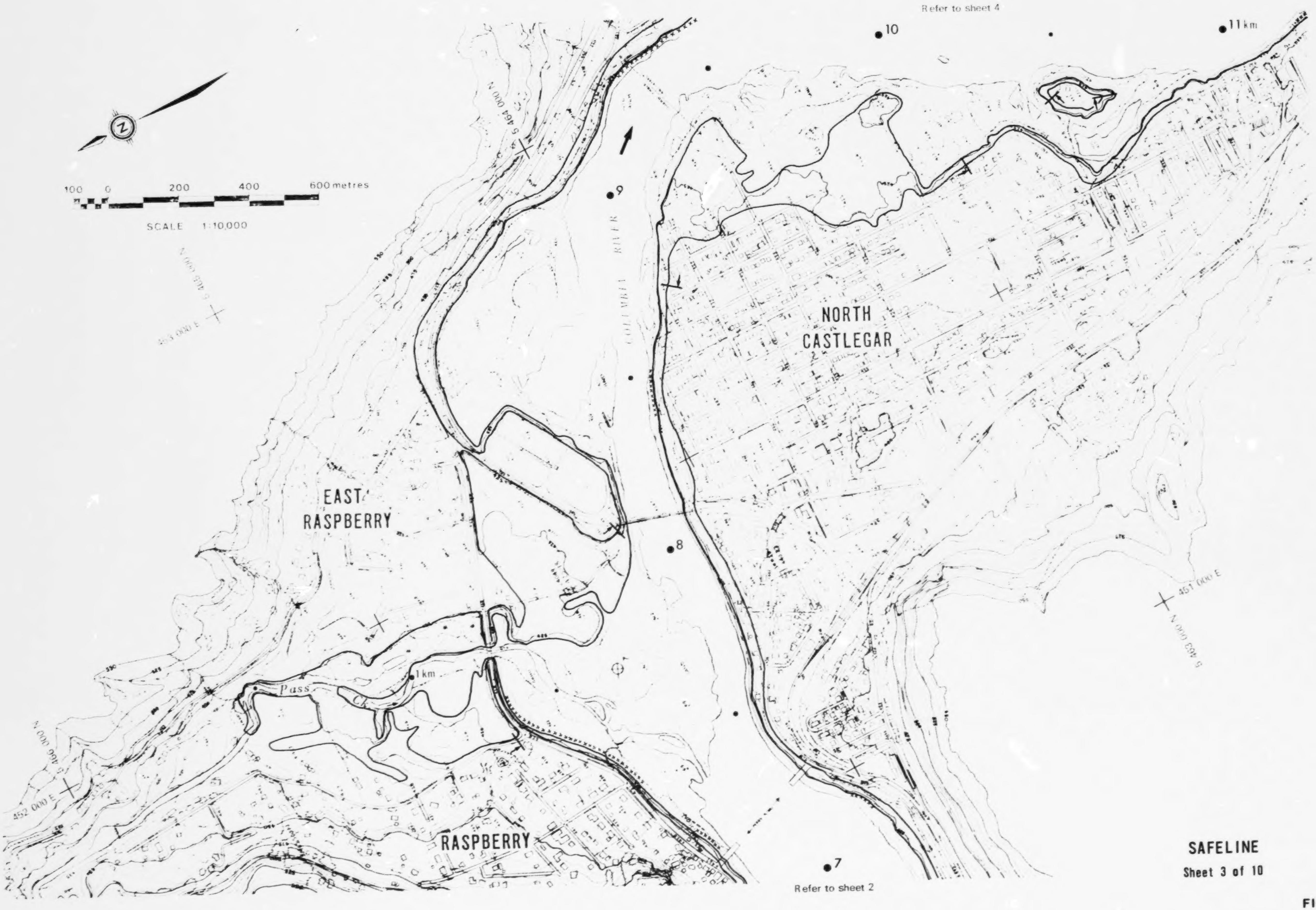
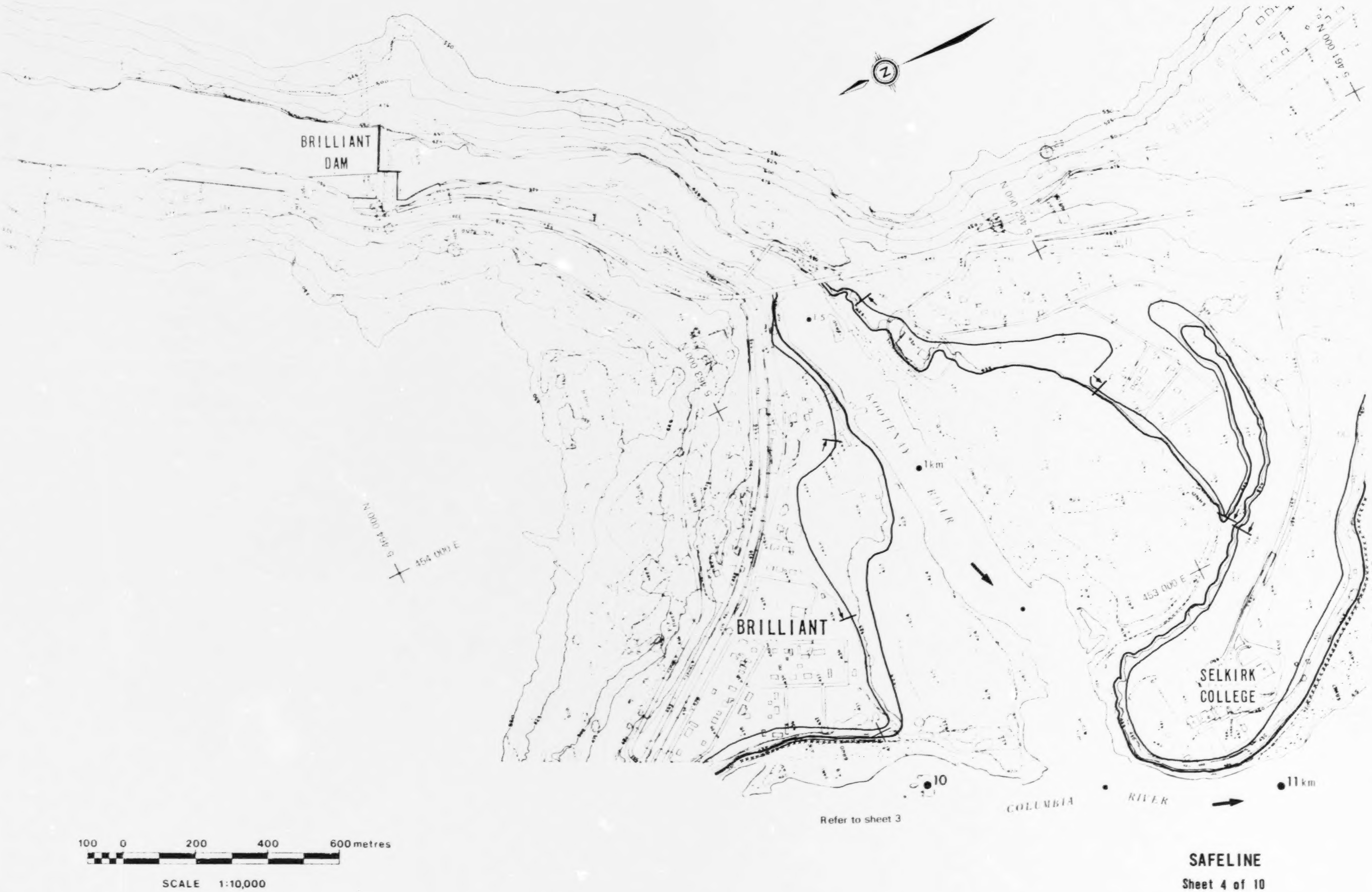

Refer to sheet 4
10
11km
9
COLUMBIA RIVER
NORTH CASTLEGAR
EAST RASPBERRY
8
Pass Cr.
1km
RASPBERRY
7
Refer to sheet 2
100 0 200 400 600 metres
SCALE 1:10,000
5 464 000 N
453 000 E
451 000 E
452 000 E
SAFELINE
Sheet 3 of 10

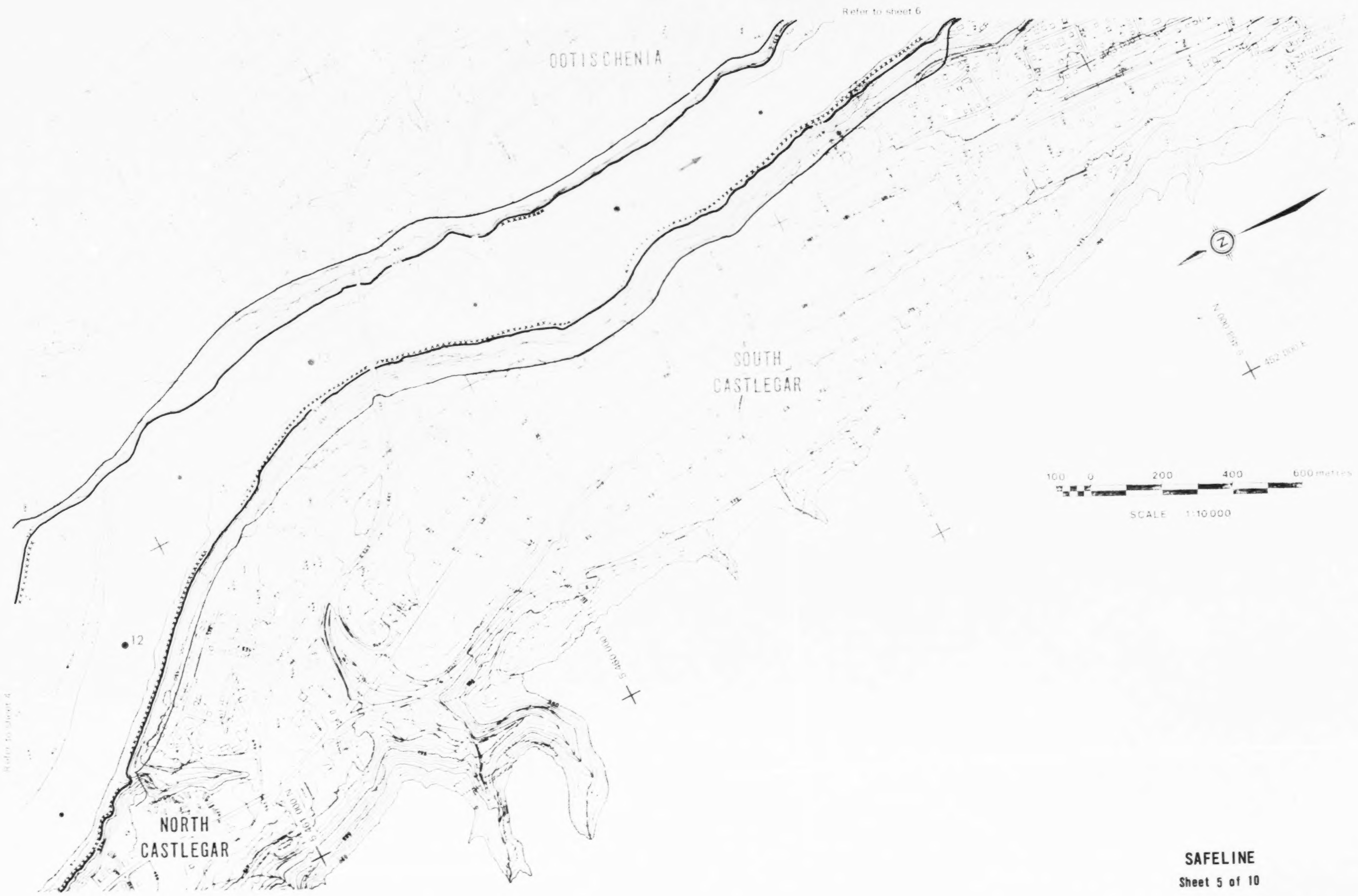
FIGURE C-2



**SAFELINE**

**Sheet 4 of 10**

**FIGURE C-2**



**SAFELINE**

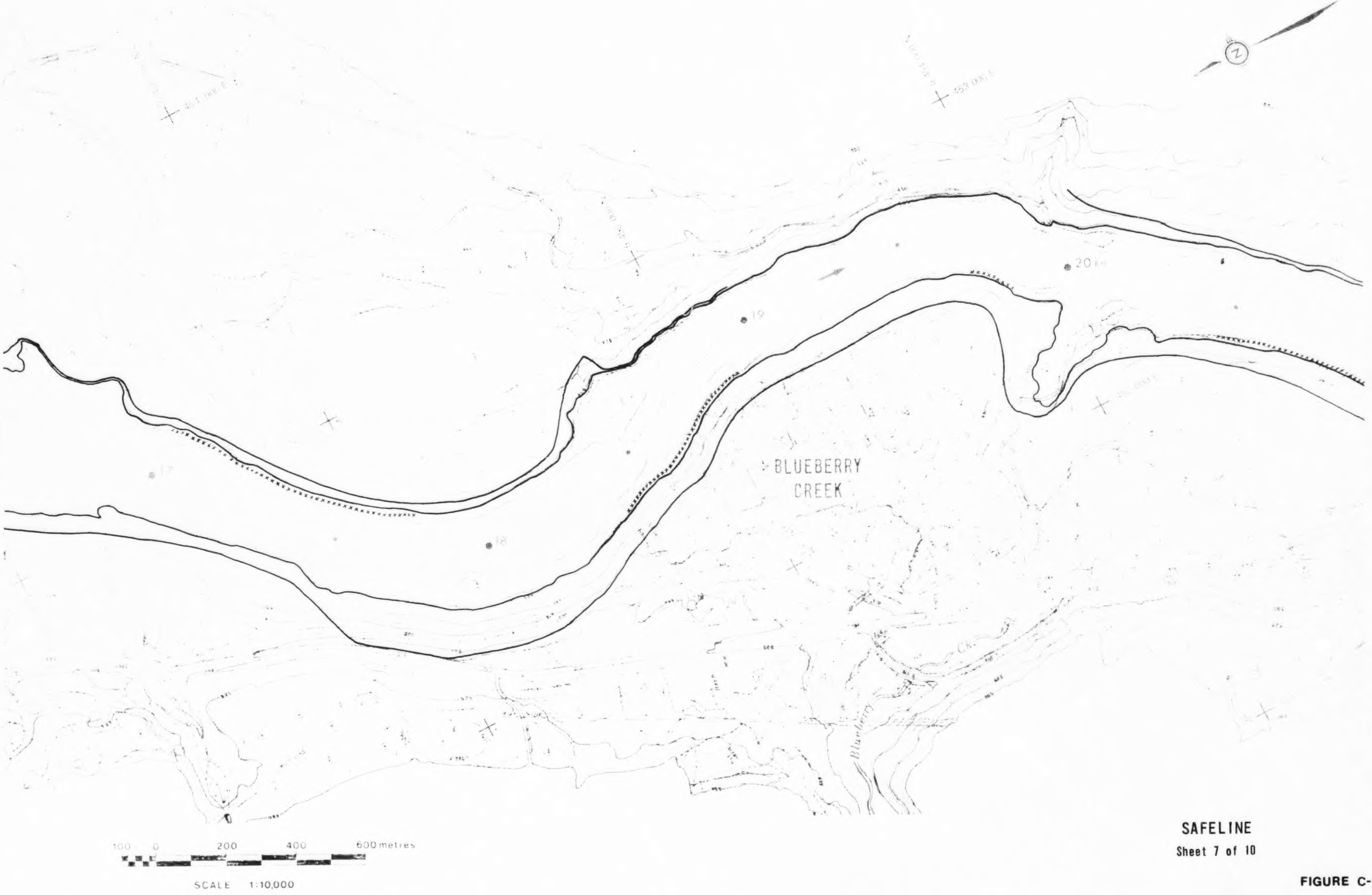
**Sheet 5 of 10**

**FIGURE C-2**

**SAFELINE**

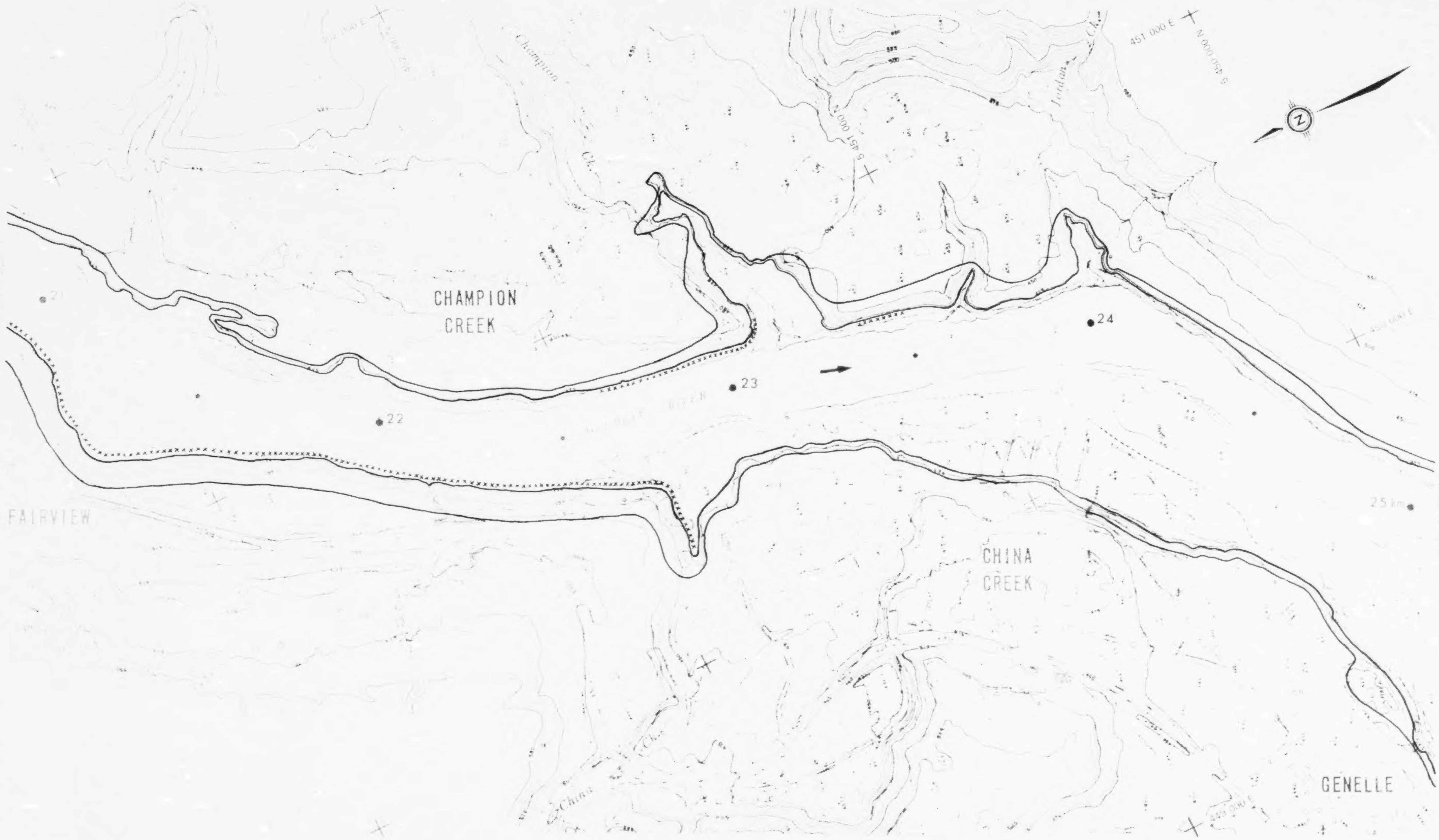
**Sheet 6 of 10**

**FIGURE C-2**



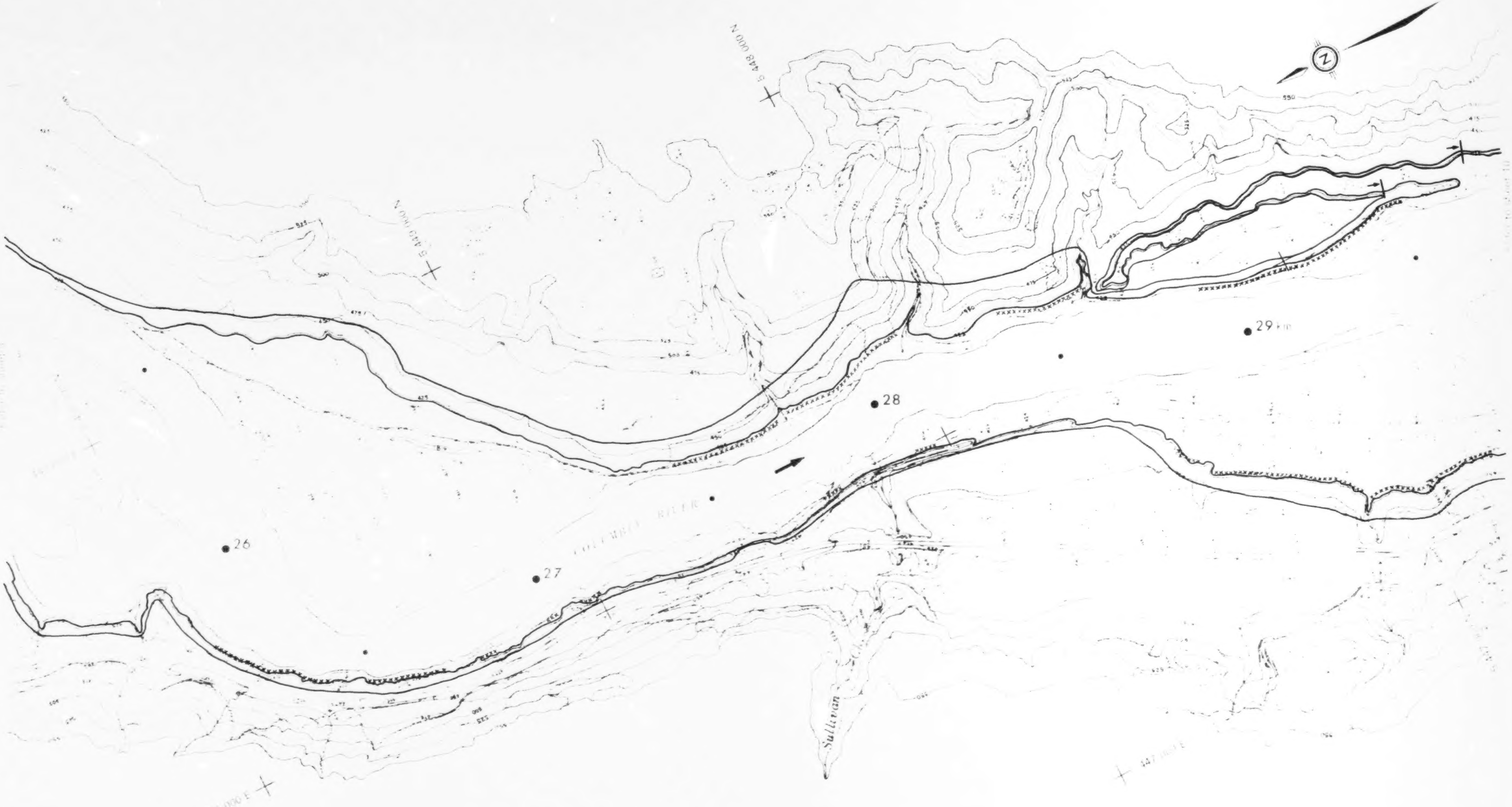
SAFELINE
Sheet 7 of 10

FIGURE C-2



**SAFELINE**
Sheet 8 of 10

**FIGURE C-2**



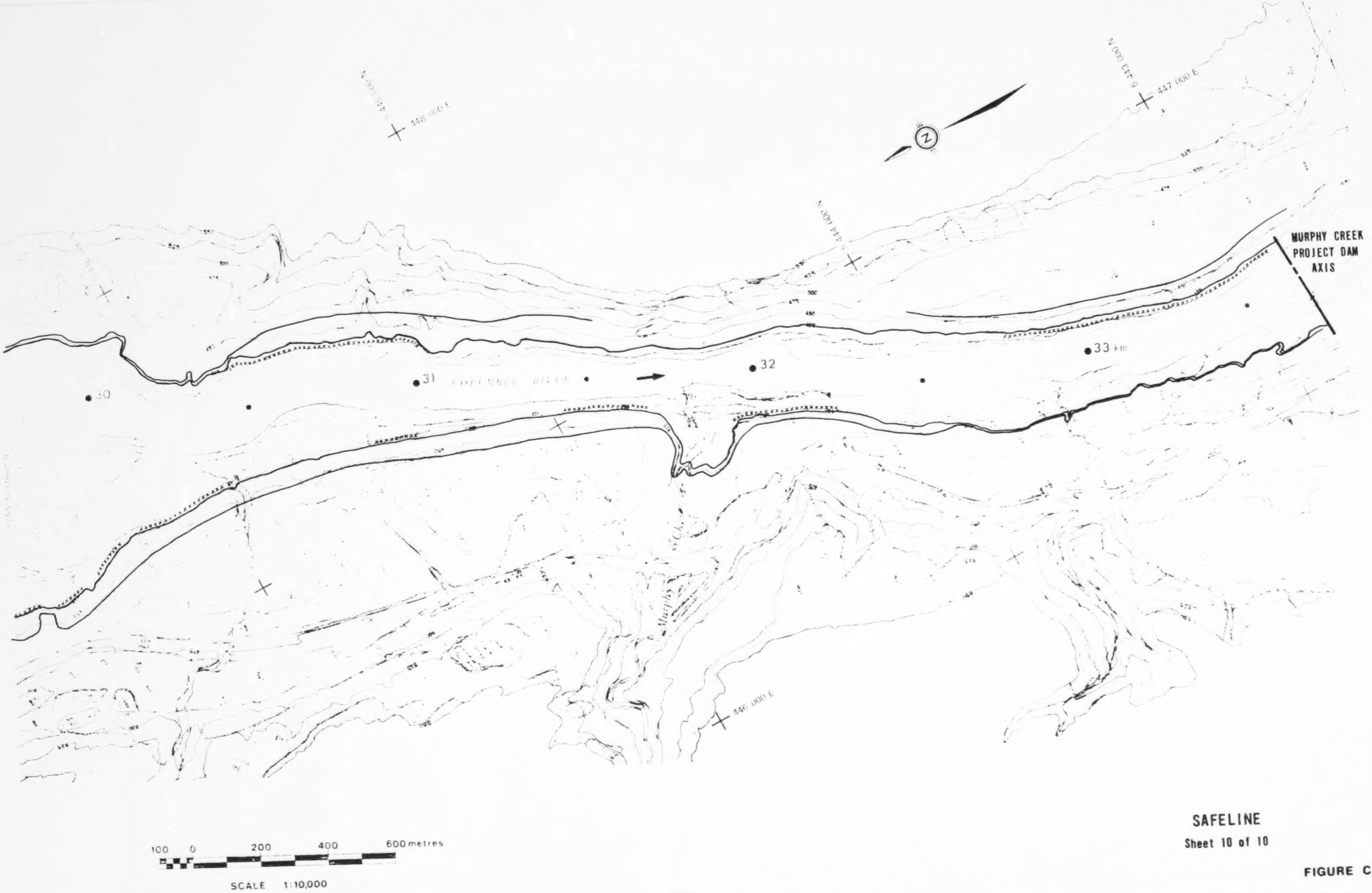
100 0 200 400 600 metres

SCALE 1:10,000

**SAFELINE**

**Sheet 9 of 10**

**FIGURE C-2**



**SAFELINE**

Sheet 10 of 10

**FIGURE C-2**

POOR

QUALITY

ORIGINAL